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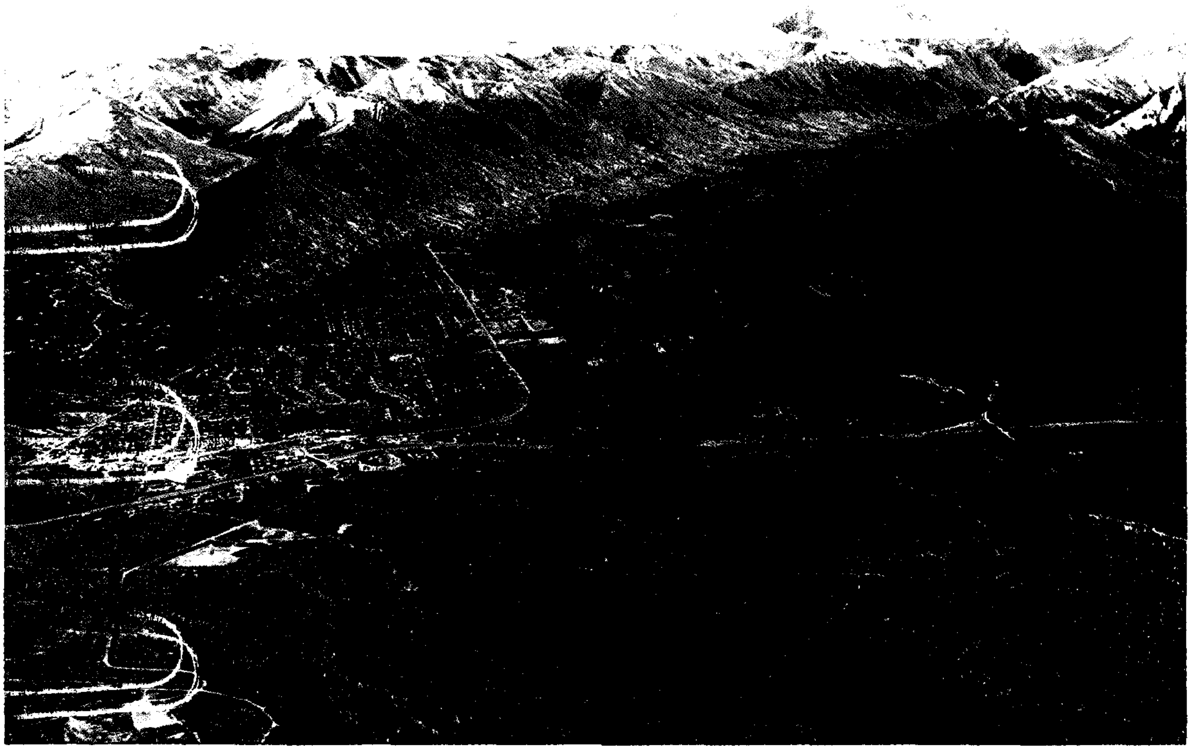
TASK 2
PRELIMINARY DAMSITE INVESTIGATION

Appendix

Eagle River Water Resource Study



Municipality of Anchorage
Water and Sewer Utilities

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CH2M HILL

December 1981

TASK 2
PRELIMINARY DAMSITE INVESTIGATION

Appendix

II

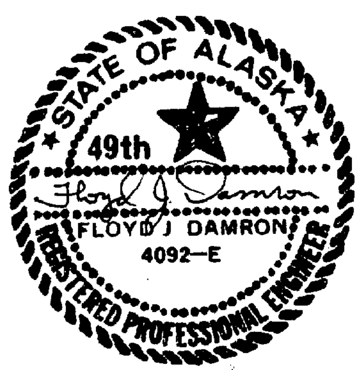
Eagle River Water Resource Study

Municipality of Anchorage
Water and Sewer Utilities

CH2M HILL

December 1981

Cover photo by: Air Photo Tech, Inc.



This report was prepared under the supervision of a registered professional engineer.

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PREFACE

To pursue the recommendations for further study that were prescribed in the Metropolitan Anchorage Urban Study, completed by the U.S. Corps of Engineers in 1979, the Municipality of Anchorage engaged CH2M HILL to conduct the Eagle River Water Resource Study. The purpose of the study is to investigate the potential sources of water supply from the Eagle River Valley. The original scope of the study comprised four tasks:

Task 1	Well Drilling Program
Task 2	Preliminary Damsite Investigation
Task 3	Flour Water Treatment Study
Task 4	Transmission Main Design

Task 5, Eklutna Lake Alternative Water Source Evaluation, was added to the scope after the completion of the first four tasks.

The report for each task is bound separately and is an appendix to the Executive Summary of the entire study. This Appendix II is the report for Task 2, Preliminary Damsite Investigation.

ACKNOWLEDGMENTS

We were assisted throughout the study by the Anchorage Water and Sewer Utilities staff.

Dr. Robert Carlson, Director of the Institute of Water Resources, University of Alaska at Fairbanks, provided assistance on the flood frequency, sedimentation, and ice analyses.

Eklutna, Inc. provided pertinent information at the weekly meetings and ready access to its property.

Exploration Supply and Equipment, Inc., provided the drilling for the field exploratory borings.

Harding-Lawson Associates, Anchorage, Consulting Engineers and Geologists, conducted part of the laboratory testing on the soil samples.

Lindvall, Richter & Associates, Los Angeles, prepared the preliminary seismic evaluation, Exhibit C, for this project.

Air Photo Tech, Inc. provided the aerial photographs.

We wish to express our appreciation to these people and organizations for their assistance and for their cooperation during the course of this study task.

SUMMARY AND CONCLUSIONS

The purpose of the Eagle River Water Resource Study, which consists of five separate tasks, is to study the potential of developing the Eagle River Valley as a source of municipal and industrial water supply for the Municipality of Anchorage. Task 2, the subject of this report, is a preliminary damsite investigation for use in determining the feasibility of developing Eagle River as a surface water supply source.

Two damsites were identified for study. Preliminary investigations were conducted for each site to determine the size of dams that would form reservoirs capable of meeting a constant diversion of 73 cfs (47 mgd) and 108 cfs (70 mgd). Based on these analyses, the lower damsite, located 1-1/2 miles east of the Glenn Highway bridges, was chosen as the preferred site. These analyses also indicated that it would be more practical to construct the dam to provide the ultimate desired water supply demand rather than attempting to stage construction as the water supply demand increases.

A dam can be constructed at the preferred damsite to meet a water supply demand of up to 108 cfs, provided there are no major deviations from the following assumptions used in the investigation:

- 31 cfs (20 mgd) is adequate for minimum downstream releases
- Mitigation of fisheries could be achieved to the satisfaction of controlling agencies
- Other environmental impacts would not prohibit construction of the dam or use of water from the river
- Special interest groups would not intervene to block construction of the dam or water use
- Sediment deposition in the reservoir could be minimized by draining the reservoir during the summer when the sediment load in the river is the highest
- All permits and licenses could be obtained from the appropriate agencies

A constant water demand of 108 cfs was used to prepare the conceptual design of the dam and appurtenances. The conceptual dam design was prepared only for the preferred damsite, as identified by the Anchorage Water and Sewer Utilities. The proposed Eagle River dam would be constructed of compacted earth fill, approximately 80 feet high, with a crest length of about 800 feet. The embankment would have a nominal crest elevation of 350 feet, National Geodetic Vertical Datum (NGVD). The normal pool surface would be at elevation 338 feet, with a reservoir surface area of 2,530 acres, and a total storage volume of approximately 55,000 acre-feet. The maximum pool surface, achieved only under the most critical flood conditions, would be at about elevation 344.5 feet, with a reservoir area of approximately 2,840 acres and a total storage volume of 71,200 acre-feet. The spillway would be a reinforced concrete chute with a horizontal apron stilling basin. The spillway discharges would be controlled by three 30-foot-square radial gates. Two 10-foot-square low-level outlet conduits would be provided for reservoir drainage and summer sediment bypassing, and a 3-foot-diameter outlet pipe would provide water for minimum streamflow and for fish facilities.

The low-level outlet gates would be open during the summer and the reservoir would be near empty to allow passage of the high sediment-laden river flows. During late August the low-level outlets would be closed to begin storing water for later use. The minimum downstream releases would be met at all times. The reservoir normally would fill by mid-October and would be drawn down as needed to meet the water supply demand during the winter and spring. On or about May 1, when river flows are sufficient, the low-level outlets would be opened and the reservoir drained.

Based on our hydrologic, geologic, and geotechnical analyses, the proposed dam can be constructed to safely withstand the maximum credible earthquake and the probable maximum flood.

This dam is estimated to cost $23,240,000 in April 1981 dollars. This amount is for construction and engineering only, and does not include land acquisition, financing, escalation to a future construction date, or fish facilities.

Many uncertainties encountered during the preliminary damsite investigation need to be resolved by additional studies before any dam on the Eagle River can be designed. If the Anchorage Water and Sewer Utilities decides to proceed with design of the Eagle River dam, we recommend the following studies or actions be completed prior to or during design:

- o Determine the type, number, migration pattern, and distribution of fish in the river
- o Determine the potential effect of the old Eagle River dump on reservoir water quality and evaluate any modifications required to develop the Eagle River as a water source

- Study the variability of the water supply demand throughout the year

- Install at least one precipitation station in the upper reaches of the basin to provide hourly precipitation data, and modify other stations in and around the basin to provide hourly precipitation data

- Recompute the probable maximum flood during design

- Study the winter regime of Eagle River

- Perform a more detailed seismicity study for the site

- Perform additional subsurface exploration surrounding the damsite

- Perform additional borrow exploration and testing for select materials such as core and filter materials

- Study the effect of not stripping vegetation from the reservoir

- Conduct a sediment sampling study to provide data for reservoir sedimentation estimates

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Chapter 1
INTRODUCTION

BACKGROUND

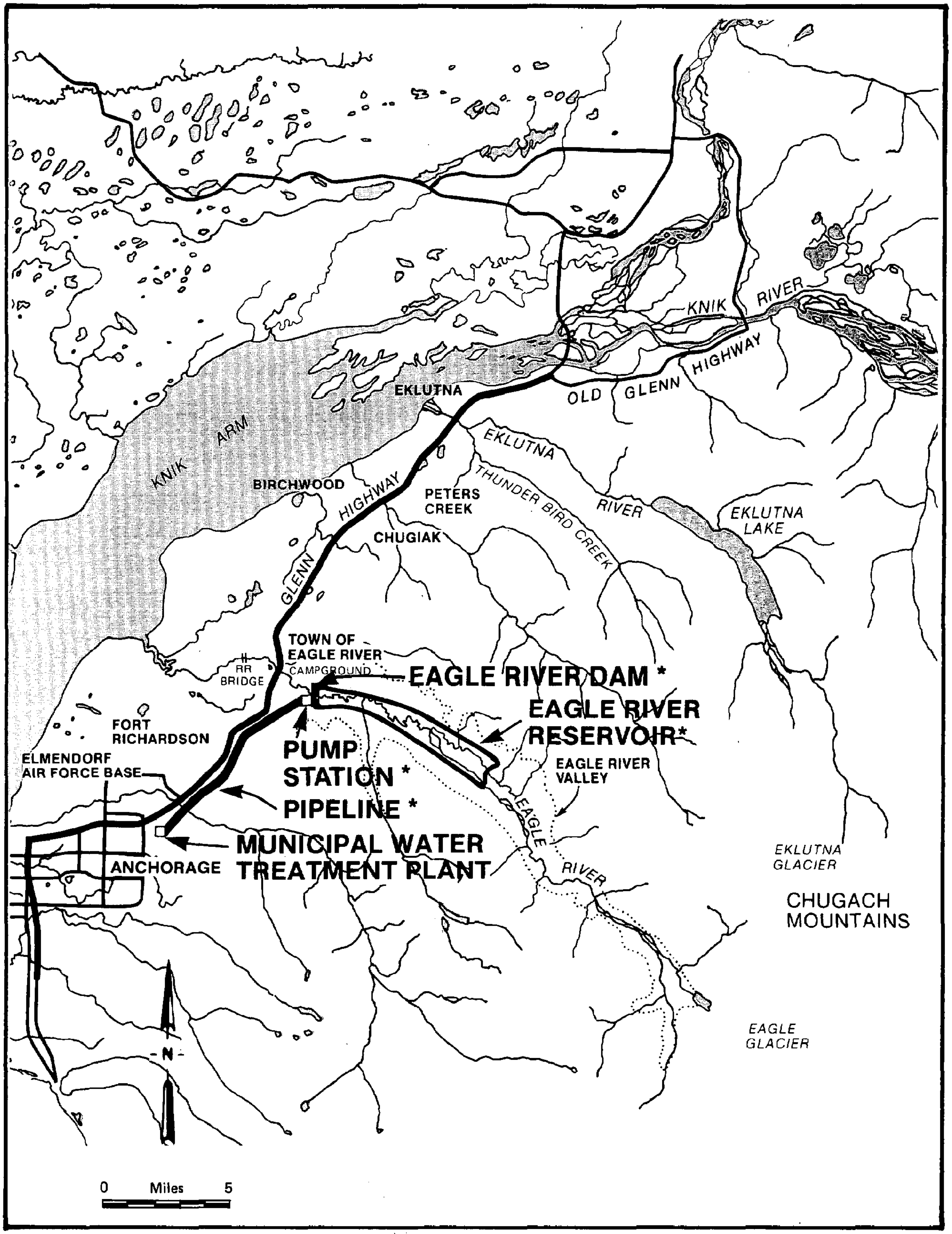
The population and, thus, the water supply needs of the metropolitan Anchorage area are rapidly growing. Presently, water to Anchorage is principally supplied by surface water from Ship Creek and by groundwater wells in the Anchorage Bowl. However, if present growth trends continue, these sources will not meet future needs.

In 1974 the United States Congress authorized the U.S. Army Corps of Engineers to perform the Metropolitan Anchorage Urban Study (MAUS), which was completed in 1979. The purpose of the MAUS was "to evaluate the adequacy of the developed water supply in the metropolitan Anchorage area, to determine future water demands, to assess sources for water supply development, and to formulate water supply plans to meet the increased future demand" (U.S. Army Corps of Engineers, 1979). The MAUS study area comprised the Anchorage Bowl and the area northeast to the town of Eklutna (Figure 1-1).

The projected future water demand increases, determined in the MAUS, are shown in Figure 1-2. It is expected that by the year 2025 an additional 81.5 million gallons per day (mgd) of water will be needed to meet the increased demands in the area.

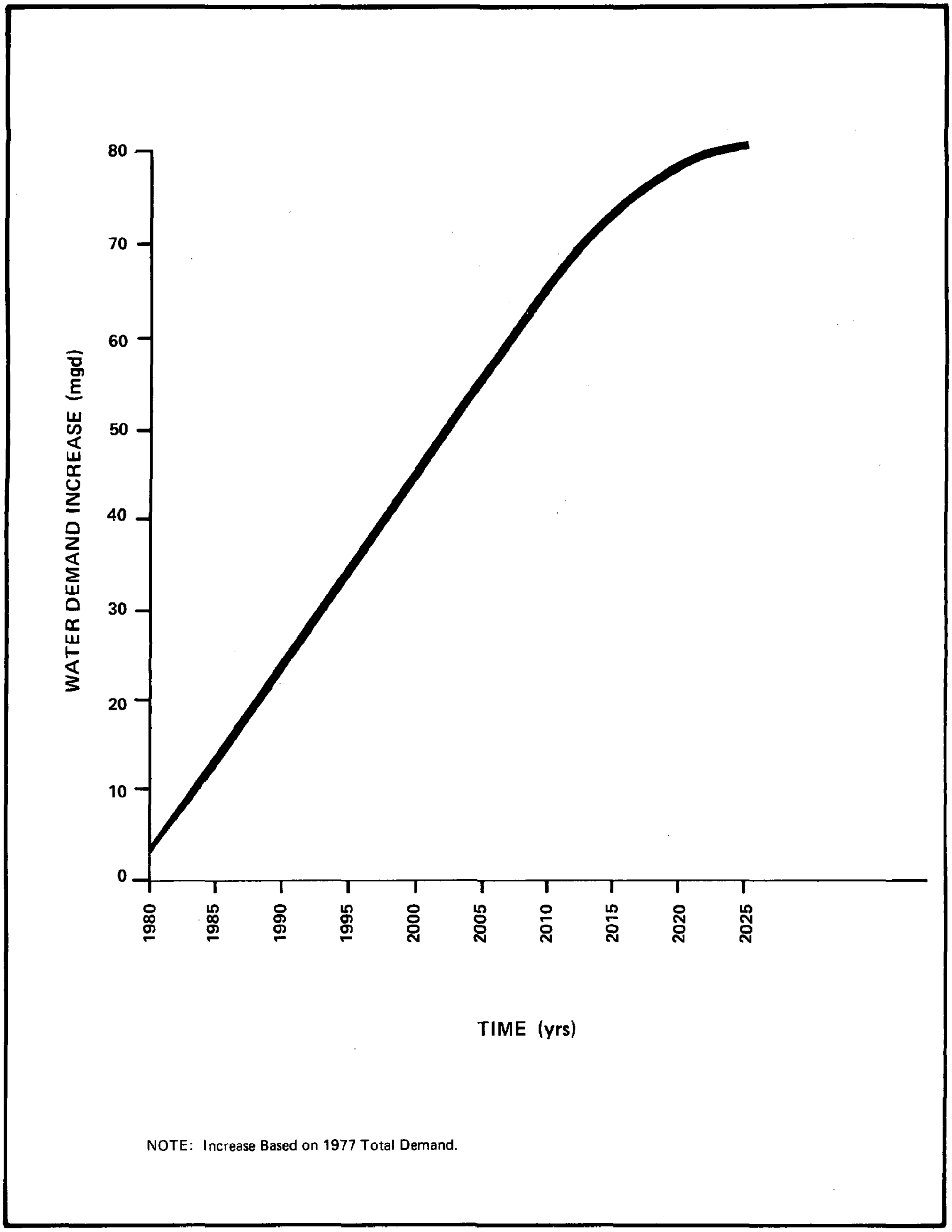
The MAUS report identified many potential sources of supply: Eagle River Valley groundwater; Anchorage Bowl groundwater; and surface water from Campbell Creek, Ship Creek, Eagle River, and Eklutna Lake. Two plans were recommended by MAUS for future study. Plan IV, which ranked first environmentally and socially, included a combination of supply from Ship Creek, Anchorage Bowl groundwater, and Eklutna Lake. Plan VI, which ranked first on an economic basis, included an increased supply from Ship Creek, winter diversion from Eagle River, further development of Anchorage Bowl groundwater, and exploration for Eagle River Valley groundwater.

To increase the existing water supply sources within the Anchorage Bowl, the Municipality recently constructed a 36-inch supply main to its water treatment plant from the military diversion facility on Ship Creek. Other developments are expected to include new wells to increase groundwater supply and the expansion of the Municipal Water Treatment Plant facilities. However, rapidly growing demands in Anchorage require development of a new source outside the Anchorage Bowl within the next 10 years. The Eagle River-Chugiak-Eklutna area, northeast of Anchorage, needs a new source now.



*As recommended in this study

Figure 1-1
Vicinity Map



SOURCE: U.S. Corps of Engineers. 1979.

Figure 1-2
Projected Water Demand Increase 1980-2025

As a result of the MAUS findings, the Municipality decided to investigate potential sources outside the Anchorage Bowl that could supply 70 mgd of water. On the basis of the MAUS population projection, this diversion would satisfy the demands of the entire study area through the year 2012. The increases in water supply capacity that are expected to be developed within the Anchorage Bowl would delay the need for the full 70-mgd capacity of the new water source outside the Bowl until approximately the year 2020 or longer.

To investigate possible sources of water supply outside the Anchorage Bowl, the Municipality engaged CH2M HILL to conduct the Eagle River Water Resource Study. This original scope of the study comprised four separate tasks to investigate the Eagle River Valley as a potential source of municipal and industrial water supply:

- Task 1, a well drilling program to study the feasibility of developing the Eagle River Valley as a groundwater source

- Task 2, a preliminary damsite investigation to determine the feasibility of developing the Eagle River as a surface water source

- Task 3, an investigation to determine if the glacial rock flour in the Eagle River water is removable by conventional treatment processes

- Task 4, preliminary design of a pipeline to transport groundwater or surface water from the Eagle River Valley to Anchorage

Each task was conducted independently.

The results of the first four tasks clearly indicate that a substantial dam and reservoir are required to develop Eagle River as a water source. Before committing itself to this dam and reservoir project, the Municipality of Anchorage increased the study scope to include Task 5, Eklutna Lake Alternative Water Source Evaluation, to analyze the capability of Eklutna Lake to supply the 70 mgd of water to the area. The lake is 30 miles northeast of downtown Anchorage and 16 miles northeast of the Eagle River (Figure 1-1).

The report of each task appears as an appendix to the Executive Summary of the entire study. This appendix is the report for Task 2, Preliminary Damsite Investigation.

PURPOSE AND SCOPE

In Task 2, a study was performed to identify the preferred site and size for a dam that would fulfill certain criteria. The Eagle River dam and reservoir are to store a sufficient quantity of water during late summer and fall to provide a constant supply of 108 cfs (70 mgd) to a treatment plant. In addition, minimum streamflow below the dam would have to be maintained. Summer withdrawals for water supply can be made from the river without significant impoundment. Therefore, the reservoir would be drained during the summer, and the river would allowed to flow near its natural level. This would minimize sedimentation in the reservoir and the dam's impact on Eagle River summer flows.

This report contains the results of field and laboratory tests, the engineering analysis of the proposed damsite, and the conceptual design of the dam and appurtenances. Included within these topics are discussions of:

- Major project objectives
- Hydrology and hydraulics
- Regional and site geology
- Preliminary evaluation of seismicity
- Environmental considerations
- Dam alignment and geometry
- Preliminary cost estimate
- Construction operation and maintenance considerations
- Conclusions and recommendations

The conceptual design of the dam, an approximate construction cost estimate, and recommendations for required studies are presented to aid the Municipality of Anchorage in comparing the Eagle River Valley with other potential water sources.

SITE DESCRIPTION

As shown on the vicinity map, Figure 1-1, the major elements of the Eagle River Water Resource Study are the proposed Eagle River dam and reservoir, pump station, pipeline, and the existing Municipal Water Treatment Plant. The location of the Eagle River water treatment plant has not yet been established.

Initially, the Anchorage Water and Sewer Utilities (AWSU) requested that two damsites be considered: the site discussed in this report and an alternative site located approximately 1 mile upstream. The locations of both sites are shown on Figure 1-2. These two sites were suggested by previous investigators (Bateman, 1948; Retherford et al., 1966; U.S. Army Corps of Engineers, 1979) as potential damsites.

At the start of this task, conceptual layouts of the dams were prepared for each site. The hydraulic structures would be similar at either site. The embankment volume at the alternative site would be approximately three times the embankment volume at the lower site. In addition, aerial photograph interpretation indicates that the Eagle River in the vicinity of the alternate site has meandered back and forth across the valley floor, reworking the flood plain sediments. This suggests that the near-surface soils may be fine-grained and loose. If so, they would be subject to liquefaction during seismic loading unless they are densified or replaced. The depth of soil influenced by the meandering of the river is unknown. At the alternative site, it also would have been necessary to excavate large summer bypass channels both upstream and downstream of the dam because of unfavorable existing river channel geometry.

These preliminary findings were presented to AWSU at a project meeting on November 4, 1980. It was suggested that the study concentrate on the lower site unless it proved to be unsuitable. AWSU concurred with this approach. In general, this report discusses only the lower damsite.

The proposed Eagle River damsite is located in Section 13, Township 14 North, Range 2 West. The site is approximately 1-1/2 miles east of the Glenn Highway bridges. The reservoir will extend upstream approximately 6 miles. A plan of the reservoir and vicinity is shown on Figure 1-3.

The drainage basin consists of a broad U-shaped valley in the Chugach Mountains. The valley walls are steep, rising to elevations of 5,000 to 6,000 feet (elevation is based on the National Geodetic Vertical Datum of 1929). The valley floor and the lower valley slopes are covered with organic topsoil and decomposing vegetative material. This area supports scattered deciduous and evergreen trees. The upper valley slopes support little vegetation. The river elevation at the damsite is approximately 270 feet. The maximum elevation in the watershed occurs at the peak of Mt. Yukla, elevation 7,535 feet. The uppermost portion of the valley is filled by the Eagle Glacier.

The areas immediately surrounding the proposed reservoir are sparsely populated. The Eagle River area has a population of approximately 6,000 and lies approximately 1 mile north of the damsite. Eagle River Campground is located about 1 mile downstream of the proposed damsite on the south bank of the river, and the Glenn Highway bridges are located about 1-1/2 miles downstream. Approximately 2 miles downstream of the proposed damsite, the Eagle River enters Fort Richardson. There is an electrical transmission line located approximately 3 miles downstream of the site, and the Alaska Railroad crosses the Eagle River approximately 3-1/2 miles downstream from the site.

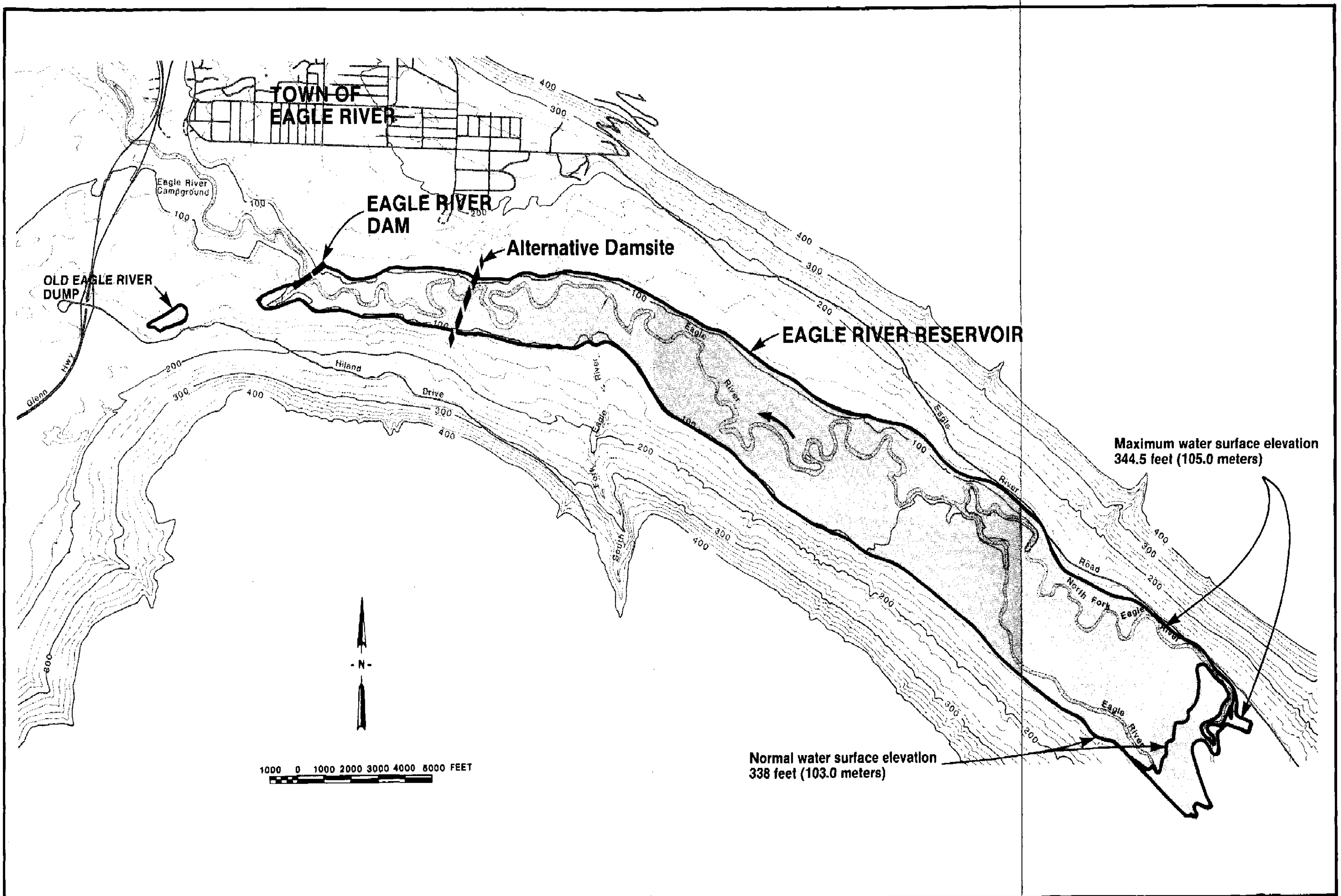


Figure 1-3
Reservoir Plan

LIMITATIONS

This report has been prepared for the use of the Anchorage Water and Sewer Utilities for specific application to the Eagle River Water Resource Study preliminary damsite investigation, in accordance with generally accepted engineering practice. No other warranty, expressed or implied, is made. In the event that any changes in the nature or location of the dam or reservoir are made, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions or recommendations are modified or verified in writing by CH2M HILL.

The analyses and recommendations presented in this report are based in part on widely spaced borings and surface observations. Variations in subsurface conditions are expected to exist between these points. The nature and extent of such variations may not become apparent until construction. When variations are encountered, it will be necessary to reevaluate the conclusions and recommendations presented in this report. The performance of an earth dam is highly dependent on the subsurface conditions, and project design must continue into the construction period.

The conceptual design presented in this report is believed to be workable, but the design concepts are not refined enough at present for incorporation into a final design. Additional investigations will be required to complete the design of the Eagle River dam.

Chapter 2
MAJOR DESIGN CONSIDERATIONS

The conceptual design of the Eagle River dam has been accomplished in accordance with commonly accepted standards for the design of earth dams and with criteria requested by the Municipality of Anchorage Water and Sewer Utilities (AWSU). This chapter identifies, in broad terms, the major considerations used in the conceptual design of the dam: safety, water supply requirements, spillway capacity and operation, seismic design, and freeboard.

SAFETY

Eagle River dam must be designed to be safe under all conditions that can reasonably be expected to occur, such as large floods and earthquakes. Conservative design details, such as extra freeboard and flat slopes, can provide a measure of safety against conditions that cannot be predicted, such as ground tilting during major earthquakes.

WATER SUPPLY REQUIREMENTS

AWSU requested that the study be conducted for water supply requirements of both 47 and 70 million gallons per day (mgd) constant flow and that consideration be given to the feasibility of building the dam in stages as the water demand increases.

All designs must also maintain minimum stream releases for the downstream fishery.

SPILLWAY CAPACITY AND OPERATION

The U. S. Army Corps of Engineers (1975) recommends that spillway design floods be based on the dam's storage capacity and on the potential hazard to property and human life downstream of the dam. Using their standards, the dam would be classified "large" because the maximum storage volume of the proposed Eagle River reservoir exceeds 50,000 acre-feet. The dam would be placed in the "high" hazard potential classification, because there are more than a few residences (temporary, at the campground) downstream of the dam that may be flooded in the event of dam failure. Also, the Glenn Highway and the Alaska Railroad bridges are of economic importance. The hazard classification is based solely on the potential for damage or loss of life downstream in the event of dam failure, and does not in any way consider the dam's condition or safety.

The Corps of Engineers recommends that the spillway design flood for a large-size, high-hazard dam be equal to the probable maximum flood (PMF). In addition, a spillway capacity adequate to safely pass the PMF is desirable because of the importance of the Eagle River reservoir in maintaining water supply to the Anchorage area.

AWSU wishes to minimize the impact of the reservoir on upstream land in the Eagle River Valley. To do this, the spillway has been designed so that under most conditions the reservoir surface will vary as little as possible above the normal water level. During the peak flow of the PMF, a maximum surcharge of about 6.5 feet is expected over the normal reservoir surface elevation.

SEISMIC DESIGN

Because of the importance of the Eagle River reservoir in maintaining water supply for the Anchorage area and the potential downstream threat in the event of dam failure, earthquake-resistant design will be a critical element of the final design process. Ancillary structures such as the spillway, gates, access bridges, outlet conduits, and the control tower must be designed to withstand the maximum credible earthquake (MCE) and still be operational. Suitable methods of structural analysis are available to do this type of design.

The embankment must be designed to limit to a tolerable amount the permanent deformation that would occur during the MCE. While there are no widely accepted standards for the earthquake-resistant design of embankments, much recent progress has been made in formulating design guidelines for potential settlement during earthquake shaking. For purposes of this conceptual design, the simplified analysis method proposed by Newmark (1965) was used to estimate the amount of crest settlement that may occur during the MCE. Additional freeboard has been provided to account for the potential settlement that may occur during seismic loading.

FREEBOARD

There are no rigid, established standards for determining the amount of freeboard required for a given earth dam; however, a substantial amount of experience exists that provides guidance in this area. In general, freeboard amounts must be determined by using the following items, which were considered in this study:

- Maximum wave height
- Maximum wave runup
- Expected crest settlement after construction
- Expected crest settlement during seismic loading
- Setup or reservoir surface tilt caused by wind
- Residual freeboard allowance to account for unknowns

Chapter 3
HYDROLOGY AND HYDRAULICS

This chapter presents the results of the hydrologic and hydraulic analyses made as part of the preliminary damsite investigation. The principal tasks performed are listed below:

- Review the existing data and reports on the river and drainage basin
- Determine the peak flows for floods with 10-year, 100-year, and 1,000-year recurrence intervals
- Develop flood profiles and identify flood limits for floods with 10-year, 100-year, and 1,000-year recurrence intervals
- Determine the low flow frequency
- Determine the spillway design flood (PMF)
- Consider potential reservoir sedimentation and ice effects
- Perform a firm yield analysis and reservoir operation study
- Develop the hydraulics for the spillway and outlet works
- Assess the possibility of developing hydroelectric generation capacity as a part of the project

BASIN DESCRIPTION

The U.S. Geologic Survey (USGS) established a stream gage on the Eagle River in October 1965 (USGS Station No. 15277100). This gage is 0.6 mile upstream of the Glenn Highway bridges and 8 miles upstream of the river mouth. The elevation of the gage is 250 feet.

The average basin elevation upstream of the gage is 2,600 feet, with several peaks over 7,000 feet. Raven Creek and the North and South Forks flow into the Eagle River above the gage.

The Eagle River drainage basin, upstream of the (USGS) stream gage, consists of about 192 square miles of glaciers, steep mountain slopes, and a 1/2- to 1-mile-wide valley floor. This basin is 27 miles long and averages 7 miles in width. A map of the basin is shown on Figure 3-1.* The head of the basin is 34 miles east-southeast of Anchorage and 33 miles southeast of the mouth of the Eagle River. The mouth is 10 miles northeast of Anchorage.

According to 1960 edition 1:63360 scale USGS maps, 17 percent of the basin is covered by glaciers. Eagle Glacier comprises 60 percent of the total glacial area. Numerous named and unnamed smaller glaciers make up the remaining area. These include Raven, Clear, Organ, and Flute Glaciers. Eagle Glacier terminates at a small lake 35 river miles upstream from the mouth of the Eagle River at an elevation of 875 feet.

The river is braided along the upper reach and meanders along the lower reach to the gaging station. The slope of the channel bottom varies from quite steep (1.4 percent) near the glacier to moderately steep (0.1 percent) between the North and South Forks of Eagle River. The surface of Eagle Glacier has slopes that range from 13.5 percent to 158 percent.

Two damsites (Figure 1-2) were considered initially for this study. The lower damsite is 1 mile upstream of the gage and the upper damsite is 3 miles upstream of the gage, measured along the river channel. The two damsites are about one mile apart in a linear measure. The area of the drainage basins of the two dams differs by only 4 square miles. The basin above the stream gage is one square mile larger than the lower damsite basin. The drainage area difference between the gage and the upstream damsite represents only 2 percent of the total drainage area. This difference is not significant, since there is less precipitation at the lower end of the basin due to orographic effects. The hydrologic analysis therefore assumed that the same flows occur at both damsites as at the stream gage.

CLIMATE

The Eagle River Basin is located in the transition zone between marine and continental influences. The climate of the basin is moderated by the Chugach Mountains, which act as a partial barrier to moist marine air from the south and east. Temperatures are moderated by the marine influences and precipitation is moderate to heavy. The upper end of the basin receives the greatest amount of precipitation due to its high elevation and proximity to Prince William Sound, through which many of the Gulf of Alaska storms pass. Mean annual precipitation ranges from about 20 inches at the USGS stream gage to more than 100 inches at the head of the basin.

*Because of the large number of figures in this chapter, they have been placed at the end of the chapter so as not to impede the flow of the text.

Precipitation averages about 40 inches per year in the drainage basin. More than half of the precipitation occurs in the period from July through October. Snow can fall at any time of the year in some parts of the basin. On the average, about 190 inches of snowfall occurs in the Eagle River Basin.

The air temperature in the basin is estimated to average about 10 degrees Fahrenheit (F) cooler than the temperature at Anchorage. The valley at the reservoir site experiences colder temperatures than Anchorage because of the glaciers and shading by the mountains. The average annual temperature of the basin is probably about 25 degrees Fahrenheit. The coldest months are generally January and February, and July is normally the warmest month. The average daily temperature in the basin is expected to be below freezing from mid-September through mid-May.

STREAMFLOW

Records taken at the USGS streamflow gage station are good except for those taken during the winter months, which are fair.

Historical Records

Streamflow records indicate that flow variations on an annual basis follow a fairly consistent pattern. Figures 3-2 through 3-5 are plots of the daily discharge hydrographs for the period from October 1 through September 30 of water years 1966 through 1980 at the Eagle River gage. The annual average flow, peak flows, and 30-day low flow are identified on these plots. Flows from November through May are usually low because of low temperatures and light precipitation. Summer runoff results from a combination of snow melt, glacial melt, and precipitation, and constitutes the bulk of the annual runoff. Summer runoff usually begins in June. Flows normally recede in August and September as cooler temperatures reduce glacial melt. The average flow in September and October is much lower than the average summer flow, but a few days of high discharge normally occur due to heavy precipitation.

The mean annual discharge of Eagle River at the gage for water years (October 1 through September 30) 1966 through 1980 is 524 cubic feet per second (cfs). The annual flows have varied from a low of 394 cfs in water year 1973 to a high of 709 cfs in water year 1967. Monthly flows are listed in Table 3-1. A diagram summarizing the variability of the monthly flows is shown in Figure 3-6.

Daily flow duration curves were plotted for the Eagle River and Ship Creek for the entire period of record at the Eagle River. These plots are shown in Figure 3-7. These plots show the percent of time that a given flow is expected to be equaled or exceeded.

Table 3-1
AVERAGE DISCHARGE FROM THE EAGLE RIVER

Water Year	Average Monthly Discharge (cfs) Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	July	August	Sept	Mean cfs	Annual mgd
1966	425	162	83	52	50	50	51	235	1,150	1,858	1,958	1,177	608	393
1967	342	103	77	67	51	53	75	255	1,507	2,116	2,221	1,593	709	458
1968	288	133	82	72	64	61	63	356	961	1,775	1,450	476	485	313
1969	155	107	70	39	32	42	78	322	1,252	1,564	874	457	418	270
1970	707	174	119	99	90	85	78	218	739	1,303	1,241	588	457	295
1971	181	103	75	57	52	40	36	82	725	1,772	2,002	552	478	309
1972	191	100	90	65	39	36	59	145	689	1,747	1,589	970	479	309
1973	418	143	123	74	48	44	71	160	662	1,290	1,227	430	394	255
1974	267	86	54	39	26	40	77	272	921	1,472	1,489	1,141	493	319
1975	231	123	81	48	44	44	77	313	746	1,652	1,307	756	456	295
1976	237	91	65	56	53	46	64	177	889	1,615	1,386	900	467	302
1977	307	190	130	94	79	62	78	249	1,333	2,120	2,424	1,100	686	443
1978	460	158	137	118	97	79	72	246	816	1,447	1,528	971	614	332
1979	324	116	72	65	61	69	117	366	1,082	2,001	2,103	1,098	618	399
1980	556	219	123	109	97	78	124	304	1,147	2,281	1,724	930	644	416
Mean	339	133	92	70	59	55	75	247	974	1,734	1,635	876	524	339
Percent of Annual	5.4	2.1	1.5	1.1	0.9	0.9	1.2	3.9	15.5	27.6	26.0	13.9	100	--
Minimum	155	86	54	39	26	36	36	82	662	1,290	874	457	394	255
Maximum	707	219	137	118	97	85	124	366	1,507	2,281	2,424	1,593	709	458

Source: USGS Water Resources Data for Alaska, 1966-1980.

Seepage Losses Along The River

Three seepage investigations along the Eagle River have been conducted by USGS. These were made during periods of relatively constant flow on April 29, 1970, May 8, 1974, and October 24, 1974. The April 1970 investigation was the most extensive, covering over 31 miles of the 35-mile river, and the October investigation covered a 16-mile central portion of the river.

Some gains and losses in river flow are noted in the published data; however, the data might be erroneous and cannot support any conclusions about river flow. For example, one of the investigations near the center of the drainage basin indicates a section of the river losing 37.9 cfs to seepage. However, this area has at least one side channel and at least one interconnecting channel with the North Fork. Flow in these channels, rather than seepage, may account for at least part of the loss in measured flow from the main river channel.

The potential for seepage from the reservoir and in the vicinity of the dam is discussed in Chapter 8.

Flow Synthesis

It is not reasonable to assume that the Eagle River stream gage has recorded the most critical (dry) year that may be experienced by the proposed reservoir during its life. Because only 14 years of recorded streamflow were available for the Eagle River gage at the time of the study, an attempt was made to correlate and extend the record on the basis of nearby gaged streams with longer records. Ship Creek and Peters Creek records were used because these streams are adjacent to the Eagle River Basin and have streamflow records overlapping the Eagle River gage record. Monthly flows for these three streams were correlated through use of the Corps of Engineers HEC-4 stream flow simulation computer program (U.S. Army Corps of Engineers, 1971.) Monthly correlation coefficients were very low for all three streams. Low correlation was expected because of the influence of Eagle Glacier.

The average annual flow per square mile at the Eagle River gage was found to be about twice as high as at Ship Creek and Peters Creek. This is probably due to greater precipitation at higher altitudes. Flow variations on an annual basis correlated very well between Ship Creek and the Eagle River. However, they did not correlate well between Peters Creek and the the Eagle River. Therefore, Peters Creek was eliminated from further consideration.

Climatological records at Anchorage are available from 1916 to the present (U.S. Department of Commerce, 1916-1979). Temperature and precipitation records were used to construct cumulative plots of annual precipitation, annual temperature, December-through-March average temperature, and February average temperature.

These plots were used as general flow predictors for the Eagle River since they indicate groupings of years that are either cooler and/or drier than normal. Records show that low temperatures, particularly in the winter, are associated with less than normal annual flow. Obviously, periods of low precipitation also indicate below-normal flows. The best predictor for low flows was found to be the December-through-March average temperature.

Additional years of streamflow were generated based only on the statistics of the historical Eagle River gage record. The justification for using the historical records was that the cumulative temperature and precipitation plots indicate that the Eagle River gage records have been collected during a generally dry, cold period. Also, water year 1969 was the lowest runoff at the Ship Creek stream gage during the Ship Creek period of record (1947 to present). Therefore, it was assumed that the Eagle River gage has recorded at least one of the most critical (low flow) years since Anchorage and Ship Creek records began, which makes the 14 years of historical record reasonable for generating a series of flows that contain dry periods. Fourteen years of monthly flows were put into the Corps of Engineers HEC-4 program in order to generate 196 years of streamflow. Exhibit A contains the output from this program. The generated flows for the Eagle River are within the range of the historical flows. These flows were used as input for the reservoir operations analysis.

Low Flow

Low flows are very important in determining whether or not a stream is capable of meeting both the water demand and downstream environmental needs. A study of monthly low flows was made to determine how often extremely low flows may occur. Based on historical data plus about 200 years of synthesized flows, a plot of the monthly low flow frequency was made. This plot is shown in Figure 3-8. The historical data points were also plotted on this figure to show the close relation between synthesized and historical data. Based on the plot, a minimum 30-day low flow of 20 cfs or less can be expected once every 100 years on the average. A minimum 30-day low flow of 32 cfs or less can be expected once every 10 years on the average.

These data are important in establishing pre-dam conditions but have little direct effect on the ability of the proposed reservoir to meet water demand. The proposed reservoir would have several months of carryover storage and would not be sensitive solely to the 30-day low flow. However, conditions causing a 30-day low flow are likely to cause adjoining months to produce lower-than-normal flows that would affect reservoir operations.

FLOOD FREQUENCY

An estimate of the flood frequency for the Eagle River was performed by Dr. Robert Carlson of the Institute of Water Resources, University of Alaska at Fairbanks. His findings were presented in a letter report to CH2M HILL dated August 1, 1980. The following is a summary quote from his study:

> A brief review of available streamflow, precipitation and snow course records indicated that insufficient data exist for a sophisticated analysis attempting to relate many of the hydrologic variables of the basin.

Instead, the Ship Creek, Eagle River and Peters Creek flow records were used to determine regional flood frequency relationships. Ship Creek has the longest record, while the Peters Creek record is almost too short to be of value. Other streams were investigated but were discarded because of insufficient records. In Dr. Carlson's opinion:

> The flow records are considered to represent a homogeneous base in both space and time. Any differences (that) the statistical analysis may show should be explainable by observable basin characteristics.

The maximum instantaneous discharge for each year was chosen for use in determining the flood frequency relationship for each gage. Flow data were plotted on log-normal probability scales for the Eagle River, Ship Creek, and Peters Creek. Due to the paucity of data, a graphical rather than analytical method of analysis was used to fit the flood frequency curves on the plotted data sheets. A straight line fit was used for all three streams.

The flood frequency curve for the Eagle River was studied on a regional basis using several different parameters. Annual floods versus the average annual flow, the drainage area, and the mean annual flood were each investigated for the Eagle River, Ship Creek, and Peters Creek. The mean annual flood versus annual flood relationship was determined to provide the best regional indicator of flood frequency because of the distribution of plotted points.

To determine the regional flood frequency curve, the average flood for the period of record for each of the streams was determined. A plot was then made using the theoretical flood frequency curve divided by the average flood for each basin. All three curves were plotted on a single figure. A graphical analysis of this plot was done to determine the regional flood frequency curve of the Eagle River. Figure 3-9 shows a plot of this curve along with a plot of the historic flood data from Eagle River. A summary of flood frequencies is shown in Table 3-2.

Table 3-2
EAGLE RIVER FLOOD FREQUENCY

Frequency (Years)	Flood Flow (cfs)
10	5,200
100	7,600
1,000	10,000

FLOOD PROFILES AND BOUNDARIES

The hydraulic characteristics of the Eagle River were analyzed to provide estimates of flood profiles for the 10-year-, 100-year-, and 1,000-year-recurrence-interval floods. This was done to permit a comparison of natural flood limits with flood limits that would result from dam construction. Water surface elevations were computed using the HEC-2 step backwater computer program (U.S. Army Corps of Engineers, 1979).

Cross sections for the backwater analyses were scaled from 1:2400 scale topographic maps with a contour interval of 4 feet (Municipality of Anchorage, 1978). The underwater portions of the cross sections were estimated from field observations. The locations of the cross sections used in the hydraulic analyses are shown on the flood profiles (Figures 3-10 through 3-16) and on the floodplain boundary maps (Figures 3-17 through 3-20).

Channel and overbank roughness factors (Manning's "n") for the backwater computations were assigned on the basis of inspection of aerial photographs (Air Photo Tech, Inc., 1979), aerial reconnaissance, and limited field observations of flood plain areas. Manning's "n" values ranged from 0.030 to 0.050 in channel sections and from 0.10 to 0.15 in overbank areas.

The hydraulic analyses for this study are based on the effects of unobstructed flow. The flood elevations shown on the profiles are valid only if substantial amounts of debris or ice do not collect in the flow path. Higher flood levels can be expected to result from log jams or ice jams.

The accuracy of the computed water surface profiles is limited by lack of detailed field reconnaissance and cross-section surveys. More detailed effort was beyond the scope of this study. Computed water surface elevations are expected to be accurate to within 4 feet for floods of selected recurrence intervals. This study is not intended to take the place of a detailed flood study, which would be conducted for the national flood insurance program. The flood profiles are shown on Figures 3-10 through 3-16 for both pre-dam and post-dam conditions.

The boundaries of pre-dam inundation for the 10-year, 100-year, and 1,000-year floods were plotted on the 1:2400 scale maps (Municipality of Anchorage, 1978) using the flood elevations determined at each cross section. Between cross sections, the boundaries were interpolated. These boundaries were then transferred to photo base maps at a scale of 1:12000 (Air Photo Tech, Inc., 1979). The pre-dam floodplain boundaries are shown on Figures 3-17 through 3-20. In cases where the 10-year, 100-year, and 1,000-year flood boundaries are close together, only the 100-year boundary is shown.

The approximate limits of the reservoir are shown on Figures 3-21 through 3-24. These limits are based on a normal water surface elevation of 338 feet.

Small areas both within and outside the flood boundaries and reservoir limits might lie above or below the water surface elevations shown. Because of limitations of the map scale and/or lack of detailed topographic data, all such areas cannot be delineated.

SPILLWAY DESIGN FLOOD

As stated in Chapter 2, the PMF was used as a basis for the spillway design. The PMF is the flood expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The PMF was based on the probable maximum precipitation, the basin lag time, and historic flood records.

Probable Maximum Precipitation

The probable maximum precipitation (PMP) for the Eagle River Basin was determined by methods described in the U.S. Weather Bureau's Technical Report No. 47 (U.S. Department of Commerce, 1963). The PMP is not identified with any specific month, but may be more likely to occur in August, September, or October. The 24-hour PMP produces 6.7 inches in 6 hours, 9.5 inches in 12 hours, and 11.6 inches in 24 hours. A storm more than 24 hours long was not considered because the spillway is sized to handle floods up to one-half the PMF without surcharging the reservoir (see Chapter 8). Therefore, longer duration floods would be passed directly through the reservoir without altering the flood-handling capability for the 24-hour PMP. Precipitation was plotted in the form of a depth-duration curve to aid in obtaining incremental rainfall. The patterns of historical 24-hour rainstorms from January 1953 through October 1970 were reviewed and used along with the the U.S. Weather Bureau's Hydrometeorological Report No. 43, to estimate a critical storm pattern (U.S. Department of Commerce, 1966).

Basin Lag Time

Basin lag time is a measure of a basin's response time to a storm. The basin lag is most frequently defined as the time from the centroid of effective rainfall to the hydrograph peak. The length of basin lag time has an important effect on the PMF. Lag time was estimated by a number of methods.

Several preliminary lag-time estimates were made on the basis of travel time of channel flow in the main stream only. The channel length and slope are important to this estimate. Using these parameters in empirical equations, calculations of basin lag time ranged between 3.8 and 7.1 hours. Since overland flow can increase the lag time substantially, estimates of lag time based on overland flow plus channel flow from the most distant points in the basin were also made. Lag-time values ranged from 6.0 to 9.8 hours using this method. Our experience with similar basins indicates that the lag time for the PMF would be more than the low-end estimate obtained from the empirical methods.

Historical precipitation and streamflow data were also used to estimate the basin lag time. Hourly precipitation at Anchorage and hourly streamflow records at the Eagle River gage were used as predictors of lag time. The Anchorage climatological station experiences little orographic influence, but the station can be used as a general indicator of the timing and pattern of some of the larger rainfalls. Six-hour precipitation data from Elmendorf and the South Fork of the Eagle River were also used; however, only limited data are available from those sites. The lag time predicted by historical data ranges from 16 to 30 hours. While no data from large floods were available for analysis, the shorter lag times resulted from rainstorms of higher intensity. Consequently, the PMF lag time is expected to be even less than 16 hours. A shorter lag time for the PMF can also be expected since the high discharges could result in increased flow velocities.

The lag time used for the PMF calculation was 11.5 hours. This value was chosen on the basis of our judgment, since historical records tend to provide too high an estimate, and empirical methods tend to provide too low an estimate of lag time for this type of basin.

Prior to the final design of the dam spillway, it is recommended that at least one recording rain gage be installed in the upper reaches of the Eagle River Basin. This would be of great benefit in computing basin lag time and, in turn, computing the PMF.

Derivation of the Unit Hydrograph

A unit hydrograph is an estimate of drainage basin runoff versus time that may result from a unit amount of effective rainfall (the portion that becomes runoff) occurring over a given period of time.

Several unit hydrographs were developed for use in estimating the PMF at the Eagle River. The Soil Conservation Service method, as described in Design of Small Dams (U.S. Bureau of Reclamation, 1974), was used to develop a unit hydrograph. The hydrograph analysis, based on historic floods and described in Design of Small Dams, was used to develop other unit graphs. Rainfall durations of 1/2 to 1 hour were studied.

The storm of September 11 to 19, 1978, was used as the basis of the final unit hydrograph. From the historical flood hydrograph, base flow was separated and a net hydrograph and corresponding dimensionless graph were derived. A 1-hour unit hydrograph was then calculated from the dimensionless graph using the 11.5 hour estimated lag time. A plot of the unit hydrograph is shown in Figure 3-25 and represents an estimate of the runoff that may occur at the damsite from 1 inch of net rainfall occurring over a 1-hour period. The peak of the unit hydrograph is 11,360 cfs and the time to peak is about 12 hours.

Probable Maximum Flood

In order to compute the PMF, a base flow equivalent to the 100-year flood peak was assumed for the basin. This corresponds to 7,600 cfs. This base flow is equivalent to 1.5 inches (water equivalent) of snowmelt during the 24-hour storm and is considered adequate to account for snowmelt and glacial melt conditions during the PMF. Much of the basin is very steep and rocky and would probably retain little water during the high intensity rainfall that occurs during a PMP. No infiltration losses were used since the basin was assumed to be completely saturated by antecedent storms. This assumption is valid for this level of study; using a minimum infiltration rate may decrease the flood peak by as much as 10,000 cfs.

The U.S. Army Corps of Engineers' computer program HEC-1, Flood Hydrograph Package (U.S. Army Corps of Engineers, 1973), was used to combine the PMP with the unit hydrograph and baseflow conditions to obtain the probable maximum flood. The PMF peak flood inflow to the reservoir is about 100,000 cfs. The PMF has a 100-hour volume of 134,000 acre-feet. The PMF hydrograph is shown in Figure 3-26.

Routing of the PMF through the reservoir was conducted assuming that the low-level outlets are closed and the reservoir is at the normal operating elevation of 338 feet at the start of the PMF.

The spillway was assumed to be passing the 7,600-cfs base flow prior to the PMF. In general, the spillway gates were assumed to be opened as needed to match outflow with reservoir inflow, while holding the reservoir at approximately elevation 338. However, the gate opening rate was limited to a rate that would not cause downstream river levels to rise faster than 2 feet per hour. This rate is used to allow people downstream along the river a reasonable amount of time to leave the river area. The routed peak outflow during the PMF is about 79,000 cfs. The reservoir surcharge is 6.5 feet, bringing the reservoir water surface elevation 344.5 feet. The routed outflow is shown on Figure 3-26.

SEDIMENTATION

Sediments carried by the Eagle River would be deposited in the quiet waters of the reservoir and would decrease its storage capacity. Determination of the deposition rate in the reservoir requires knowledge of stream hydraulics, stream characteristics, total sediment discharge, trapping efficiency of the proposed reservoir, and the expected reservoir operation plan.

Most of the required information is not available to make reasonably accurate estimates of reservoir capacity depletion by sedimentation. The following information is available:

- Daily stream discharges for the Eagle River for water years 1966 through 1980. The discharge measurements were made one mile downstream from the proposed damsite. (USGS, 1966-1980)
- Maps of the Eagle River Valley at a scale of 1:2400 with a contour interval of 4 feet. (Municipality of Anchorage, 1978.)
- Suspended sediment concentration and suspended sediment size distributions collected irregularly from 1966 through 1972. (USGS Water Resources Data for Alaska)

The above data are not sufficient to predict sedimentation rates in the proposed reservoir. However, these data can form the basis for estimates of the necessary parameters required for sedimentation rate calculations.

Stream Characteristics

Stream characteristics of importance to sedimentation estimates include slope, cross-sectional areas, width, depth, wetted perimeter, hydraulic radius, and streambed material types.

Streambed materials were observed in the field and generally consist of 1-inch minus material, with some lenses of silt, sand, gravel, and randomly scattered cobbles and boulders. The stream slope along the various stream segments was determined from the streambed profiles (Figures 3-10 through 3-16). The stream slope in the vicinity of the proposed Eagle River dam is about 0.3 percent. All other stream characteristic parameters are a function of stream discharge. The values of these parameters in the vicinity of the proposed dam are presented below.

Stream Hydraulics

The USGS has made stream discharge measurements for the last 15 years (1966 to 1980). The recorded Eagle River discharge varies from about 30 to 6,000 cfs. The discharge values were put into a computer program from which flood width, depth, area, velocity, and energy gradient were obtained. Values for the section nearest the proposed Eagle River dam were extrapolated on a log-log plot to obtain representative values at lower discharge rates.

Calculating sediment loads and sediment discharge as a function of only the mean annual discharge may introduce significant errors into the estimate. Therefore, it was decided to base sedimentation estimates on the mean monthly discharge values for a typical water year. The year chosen was 1978, with a mean annual flow of 514 cfs, close to the average annual recorded flow of 524 cfs. The 1978 average monthly flows varied from 72 to 1,528 cfs. Approximately 85 percent of the 1978 flow occurred during May through September, close to the average value of 87 percent.

Sediment Discharge

No total sediment discharge measurements for Eagle River have been taken. Suspended sediment load measurements were made at the stream gaging station at random intervals from 1966 through 1973. Figure 3-27 indicates the suspended sediment loads in tons-per-day as a function of stream discharge. The suspended sediment loads include both wash load and the sand portion of the bed load. Wash load is defined as sediment particles finer than 62 microns (silt, clay and colloids). Bed load is that material coarser than 62 microns (sand, gravel, cobbles and boulders). The measured suspended sediment loads are divided into wash load and bed load by determining the percentage of each on the basis of the particle size analysis of the suspended load samples, as shown in Figure 3-28. Suspended sediment loads, wash load, and measured sand portions of the bed load for the typical water year (1978) were then estimated. The bed load is estimated using the Colby (1957) method as outlined in Vanoni (1975). The total sediment load is a summation of the wash load and the bed load as a function of stream discharge. The estimate of the sediment loads carried by the Eagle River in the typical water year (1978) is given in Table 3-3.

Table 3-3
ESTIMATED WASH LOAD, BED LOAD, AND TOTAL SEDIMENT LOAD FOR A TYPICAL WATER YEAR (1978)

Month	Wash Load (Tons/Month)			Bed Load (Tons/Month)			Total Sediment Load (Tons/Month)		
	Low	Mean	High	Low	Mean	High	Low	Mean	High
Oct	840	2,370	6,590	2,020	3,130	5,740	2,860	5,500	12,330
Nov	140	410	1,140	390	660	1,170	530	1,070	2,310
Dec	150	430	1,170	400	680	1,210	550	1,310	2,380
Jan	80	240	690	250	400	780	330	640	1,470
Feb	40	100	290	140	200	300	180	300	590
Mar	30	70	210	120	190	220	150	260	430
Apr	20	60	160	120	120	180	140	180	340
May	200	600	1,660	740	1,240	2,560	940	1,840	4,220
Jun	2,160	6,070	16,880	5,670	10,290	19,680	7,830	16,360	36,560
Jul	6,430	18,270	51,460	11,320	20,710	38,440	17,750	38,980	89,900
Aug	7,200	19,840	55,800	13,210	23,620	45,040	20,410	43,460	100,840
Sep	3,440	9,840	27,060	6,150	11,370	21,630	9,590	21,210	48,690
Annual	20,730	58,300	163,110	40,530	72,610	136,950	61,260	130,910	300,060

Proposed Dam Operation Schedule

The volume of sediment trapped in a reservoir behind a dam is greatly influenced by the dam operation schedule. The preliminary operation schedule is based on water being stored and available for use from September through April. When river flows are sufficient, the low-level outlets would be opened and the reservoir would be drained. This is normally expected to occur in early May. From May through August, the outlet gates would be left open and the river would be allowed to flow close to natural conditions through the reservoir.

Trapping Efficiency

The trapping efficiency of the proposed Eagle River reservoir is unknown. For estimation purposes, three approaches were considered. First, all of the sediment carried by the Eagle River through the reservoir section was assumed to be deposited during both summer and winter regimes. Second, all of the sediment carried by the river in the summer was assumed to pass through the reservoir section and continue downstream with little deposition in the reservoir, with total deposition during the fall and winter months. The results of these two methods were judged to be quite conservative. Consequently, the Brune (1953) trap efficiency curve, based on the capacity-inflow ratio, was used to estimate the percentage of sediment trapped in the reservoir.

This method has been developed from data collected from many operating reservoirs. Consequently, it was judged to be the most realistic.

The Brune trap efficiency curve predicts that 5,000 acre-feet of sediment would be deposited in the reservoir in 40 to 198 years; 91 years is the predicted mean filling time. Therefore, 5,000 acre-feet of sediment storage was considered adequate for use in the reservoir operation studies.

Recommendations

The sedimentation rates and the filling scenarios for the proposed Eagle River reservoir have been estimated on the basis of a minimal amount of data. In order to provide a better estimate, more precise values for bed load along the reservoir section of the river are needed. Preferably, at least one year's sediment discharge should be measured. Also, a better estimate of the trapping efficiency of the proposed reservoir would increase the accuracy of sedimentation estimates. Trapping efficiency can be more accurately estimated by determining stream characteristics and stream hydraulics for a series of closely spaced cross-sections along the reservoir segment of the river. The estimates of sedimentation would also be improved by calculating sediment discharges and potential sedimentation on a daily basis instead of a monthly basis.

ICE

Dr. Robert Carlson of the Institute of Water Resources, University of Alaska, Fairbanks, assisted in identifying various potential ice effects. His findings were presented in a letter report to CH2M HILL dated October 20, 1980, and are summarized in this section.

Careful consideration of ice effects is required for any proposed Eagle River reservoir. Several major ice problems related to the reservoir must be studied in more detail during final design of a dam. These problems include the growth of ice on the reservoir surface, ice jamming in the vicinity of various structures, ice forces, and frazil ice production and accumulation.

Ice growth on the reservoir surface would cause a decrease in the available storage for water supply. Estimates of potential surface ice thickness indicate that about 4 feet of ice may develop during the colder years. With the reservoir at normal level, this would occupy about 10,000 acre-feet, a substantial volume. If the water level were lowered while the ice cover is growing, the edges of the ice cover would break and become beached before four feet of thickness is reached. Lowering the water surface would also expose less surface area to increased freezing, thus decreasing the volume of ice formed. To be conservative, 4 feet of ice was assumed to cover the full reservoir.

Spring breakup could cause problems of ice-jamming in the vicinity of the gate structures and dam. If the reservoir ice cover is fairly solid when high spring flows begin to occur, the ice could be broken up and forced against the dam structures. The force of ice flows coming in contact with the various piers and gate structures must be considered in final design.

Frazil ice production should also be carefully evaluated during final design. Large quantities of frazil ice could form in the steep upstream section of the Eagle River and in the South Fork of Eagle River. It is estimated that about 600 acre-feet of frazil ice might be produced annually. This frazil ice could accumulate under the reservoir ice cover as a hanging slush dam. Then, as the slush works its way downstream, it could eventually clog the water supply intake area. The intake structure design would need to carefully consider potential frazil ice problems.

The extent and nature of winter ice and frazil ice production is difficult to predict without further study of the winter regime of the Eagle River. A winter ice survey would be invaluable for further design of this project. Estimates made within the scope of this study are very rough and would be improved with field investigations. Some parameters that should be determined in a survey include: ice thickness variation; type of existing ice regime; and whether the river is broken, open, or incurs large ice jams. Particular emphasis should be placed on frazil ice production in the river upstream of the reservoir, especially if it extends throughout the winter season, and on the nature of breakup in the existing channel.

RESERVOIR OPERATIONS AND FIRM YIELD ANALYSIS

Reservoir operations were simulated using the computer program HEC-3, Reservoir System Analysis for Conservation (U.S. Army Corps of Engineers, 1974). Exhibit B contains the output from this program. The operation studies were used to estimate reservoir size needed to satisfy firm yield and minimum reservoir release requirements. The firm yields required to be considered in this study were 73 cfs (47 mgd) and 108 cfs (70 mgd).

The simulation was performed on a monthly routing of flows. The period included 14 years (1966-1979) of gage records for the Eagle River and 196 years of monthly stream flows simulated using the computer program HEC-4, Monthly Streamflow Simulation (U.S. Army Corps of Engineers, 1971). A net evaporation rate of zero was used throughout the operations analysis.

Physical features of the reservoir were taken from 1:2400 scale maps having a 4-foot contour interval (Municipality of Anchorage, 1978). Reservoir characteristics necessary to model the storage and release features of the impoundment include water surface elevation, storage capacity, surface area, and outlet capacity.

Reservoir sizing criteria were analyzed. A diversion of 31 cfs (20 mgd) to meet minimum reservoir release requirements was used for all months in the simulation period. The State of Alaska Department of Fish and Game, in a November 4, 1980, letter to AWSU, stated that a 20-mgd downstream discharge might be adequate if no significant fisheries resources are found below the dam (State of Alaska, 1980). The same discharge was also used for the Metropolitan Anchorage Urban Study. (U.S. Army Corps of Engineers, 1979, page 3-89). This flow requirement was satisfied prior to making any releases for water supply demands. Additionally, an inactive storage volume of 5,000 acre-feet was assumed necessary for sediment accumulation. An inactive storage volume large enough to allow a 4-foot ice accumulation on the reservoir surface was also assumed.

The initial reservoir operations analysis was performed to determine active storage requirements for a range of firm yields. It was assumed for this initial analysis that the reservoir would remain as full as possible during the year.

Subsequent analyses were performed to determine the effects on the firm yield if the reservoir is emptied during the highly sediment-laden summer river flows. This was done for two different reservoir sizes. The smaller reservoir was sized for a firm yield of 73 cfs (47 mgd) plus 31 cfs for minimum reservoir release. The larger reservoir was sized for 108 cfs (70 mgd) firm yield, plus a minimum reservoir release of 31 cfs.

A firm yield of 73 cfs for domestic use, plus 31 cfs for fish flow, requires a reservoir with an active storage capacity of 26,000 acre-feet and an inactive storage for ice and sediment accumulation of 13,300 acre-feet. This results in a normal reservoir elevation of 332 feet (Figure 3-29). Reservoir filling must begin near the last week in August to meet the demand. Operation of the reservoir, with the reservoir drained May 1 through September 1, can provide 73 cfs during 99.5 percent of the time. If the reservoir is held empty through October 1, the firm yield decreases to 26 cfs (17 mgd). However, a domestic demand of 73 cfs can be satisfied about 8 out of 9 years if reservoir filling begins on October 1 each year. Table 3-4 indicates how often a given reservoir yield would not be met with various beginning filling dates. For example, from Table 3-4 it can be seen that with an October 1 filling date the yield would be something less than 73 cfs on the average of once every 9 years. Likewise, the yield would be something less than 33 cfs on the average of once every

100 years. The breakdown of the beginning filling dates was not studied for increments smaller than 1 month. Therefore, the computer analysis showed that the reservoir filling must begin August 1 to meet a firm yield of 73 cfs. However, if smaller time increments were used they would most likely show the firm yield could be met with a beginning filling date of mid to late August.

Table 3-4
RESERVOIR OPERATIONS SUMMARY
(RESERVOIR YIELD OF 73 CFS)

Date of Reservoir Filling	Yield (cfs)[a]	Fraction of Years When Yield Not Met
August 1	73	Firm based on 200 years
September 1	73	1/200
	63	Firm based on 200 years
October 1	73	1/9
	57	1/11
	45	1/14
	33	1/100
	27	1/200
	26	Firm based on 200 years

[a]Yield is in addition to 31-cfs minimum reservoir release.

An active storage of 40,000 acre-feet and inactive storage for ice and sediment accumulation of 15,000 acre-feet are required to supply a domestic firm yield of 108 cfs, plus 31 cfs for minimum flow. This results in a normal reservoir elevation of 338 feet (Figure 3-29). The 108-cfs demand can still be met with the reservoir drained May 1 through mid to late August. If the reservoir is held empty through September 1 the firm yield decreases to 63 cfs. However, a domestic demand of 108 cfs can be satisfied about 14 out of 15 years if reservoir filling begins on September 1 each year. Table 3-5 indicates how often a given reservoir yield would not be met with various beginning filling dates.

In view of the above data, a typical yearly operation to supply a firm yield of 108 cfs plus 31 cfs for minimum reservoir release may be as follows. The low-level outlet gates would be open during the summer and the reservoir would be nearly empty. During this time of the year the river flows are greatly in excess of the water supply demand so the demand can be met with a minimum pool behind the dam. During late August or early September (depending on the year), the low-level outlets would be closed to begin storing water for later in the season. The 31-cfs minimum flow would be released at all times. The reservoir would normally fill by mid-October. The reservoir would be drawn down as needed to meet water supply demand and minimum flow requirements during the winter and spring. On or about May 1, when the operators are certain that the river flow will support the demand, the low-level outlets would be opened and the reservoir drained. The annual cycle would be complete at this time and the demand would again be met directly from the river flows.

Table 3-5
RESERVOIR OPERATIONS SUMMARY
(RESERVOIR DOMESTIC YIELD OF 108 CFS)

Date of Reservoir Filling	Yield (cfs)[a]	Fraction of Years When Yield Not Met
August 1	108	Firm based on 200 years
September 1	108	1/15
	102	1/30
	94	1/40
	74	1/100
	73	1/200
	63	Firm based on 200 years

[a]Yield is in addition to 31-cfs minimum reservoir release.

Spillway and Outlets

The 100-foot-wide spillway chute is provided to safely pass the PMF. Flow to the chute is controlled by three 30-foot-high radial gates. The spillway rating curve is presented on Figure 3-30. The low-level outlets allow for diversion during construction, lowering the reservoir in the summer to minimize the reservoir pool, adjustments of flow releases during small reservoir discharges, and possible future modifications for use in a hydroelectric development. Two 10-foot-square conduits controlled by roller gates and one 3-foot-diameter pipe penetrate the embankment near the spillway. Outlet rating curves are provided on Figures 3-31 and 3-32. The tailwater rating curve, Figure 3-33, was developed using the HEC-2 step backwater program (U.S. Army Corps of Engineers, 1979) and the river cross sections used to develop the flood profiles and boundaries. The spillway and outlets are described in more detail in Chapter 8.

Downstream Scour

Trapping of sediments in a reservoir changes the balance between flow and sediment transport capacity. Therefore, the cleaner-than-normal river water downstream of the reservoir would tend to degrade (erode) the streambed as it picks up sediments from the channel. Substantial degradation of downstream channels has occurred under these conditions below many dams. The amount of degradation that may occur downstream of the Eagle River dam cannot be reliably estimated with existing data. A bedrock outcrop occurs in the river about one and one-half river miles downstream of the dam, near the campground. This may limit channel degradation to some extent downstream of the dam. Downstream of the bedrock outcrop, degradation could occur to an unknown extent. However, the river would flow nearly uninterrupted during the summer since the outlet gates would be open. This should reduce the impact of the dam on downstream channel degradation, since the most sediment-laden flows generally occur in summer. Channel degradation should be considered in final design.

HYDROELECTRIC GENERATION POTENTIAL

The mean annual flow in the Eagle River at the damsite is roughly 4.5 times as large as the 108 cfs (70 mgd) water supply demand. Since a considerable amount of water would pass over the spillway or through the dam, a brief reconnaissance-level investigation was conducted to evaluate potential hydroelectric benefits. Since the reservoir's paramount function is to provide storage to meet water supply demand, no major modifications of the reservoir operation were considered for optimizing hydroelectric benefits.

The average monthly flow released from the dam and the average monthly heads were obtained from the water supply reservoir operations studies. Normally the reservoir would be nearly full from September through April, thus providing significant head during this period. The normal discharge at the dam from November through April would be 31 cfs (20 mgd), the minimum downstream release. According to the reservoir operation plan, the reservoir would be empty during most of May through August to allow passage of the summer flows. Since the reservoir would be drained for 4 months of the year, hydroelectric potential at the site was considered to be minimal.

A small but significant change in the reservoir operation plan was considered to improve the hydroelectric potential: the reservoir would not be drawn below the spillway crest elevation, 308 feet. The reservoir storage capacity at elevation 308 is about 8,000 acre-feet, all of which would become dead storage. Minor adjustments in dam height (about 2 feet) could be made to account for this dead storage. This change in operations would provide a minimum head of 36 feet for hydroelectric generation. Using this minimum head and the design minimum discharge of 31 cfs, performance was evaluated for an adjustable propeller-type turbine-generator. Design conditions and performance are:

Design Discharge	125 cfs
Design Head (net)	48 ft.
Design Output	430 kW
Annual Energy Production	2.46×10^6 kWh
Plant Factor	66 percent

The preliminary cost estimate for this hydroelectric development was based on the Feasibility Studies for Small Scale Hydropower Additions (U.S. Army Corps of Engineers, 1979). Costs in this publication generally are based on July 1978 costs for the 48 contiguous states. These costs were updated to reflect current conditions for the Anchorage area. Additional allowances included escalation to 1982, financing during construction, and bonding fees. A bond rate of 10.5 percent and term of 20 years was used to determine annual debt service. Full development of the 70-mgd demand was assumed. The current dollar operating cost was estimated to be 92 mills per kWh. Though this value may be high compared to current costs, it may be attractive over the life of the project. Because of this cost, an additional larger capacity turbine was not considered because power costs would be even higher.

During the beginning years of the project, the water supply demand on the reservoir would be significantly less than 108 cfs (70 mgd). The reservoir could be operated with higher downstream releases to gain additional hydroelectric benefits during the initial years of the project. Then, hydroelectric output could be adjusted as the water supply demand grows. An additional benefit of maintaining a higher reservoir elevation would be the reduction of summer pumping requirements for the water supply transmission main intake at the reservoir. Additional constant flow for generation would be available if final water supply design calls for release of the reservoir water to the river so that it can be pumped from a location below the Glenn Highway bridges. These additional benefits are not included in this investigation of hydroelectric potential.

The major disadvantage of operating with the design minimum reservoir level at spillway crest is that the sediment-laden flows would not be passed through the reservoir. Because the effects of an increased pool level during the summer months cannot be evaluated without further sediment studies based on field data, and the hydroelectric benefits appear to be marginal, the hydroelectric development is not included in the project cost estimate

(Chapter 9). However, the preliminary design of the project allows for future use of one of the low-level conduits as a penstock. Stoplog slots are provided at the upstream and downstream ends of the conduits to allow dewatering and modifications for installation of a hydroelectric plant. These modifications could be made without interrupting the water supply to the Municipality of Anchorage. A life cycle benefit/cost analysis should be performed for hydroelectric generation prior to final design.

Depending on power cost increase projections, this facility may or may not yield long-term benefits.

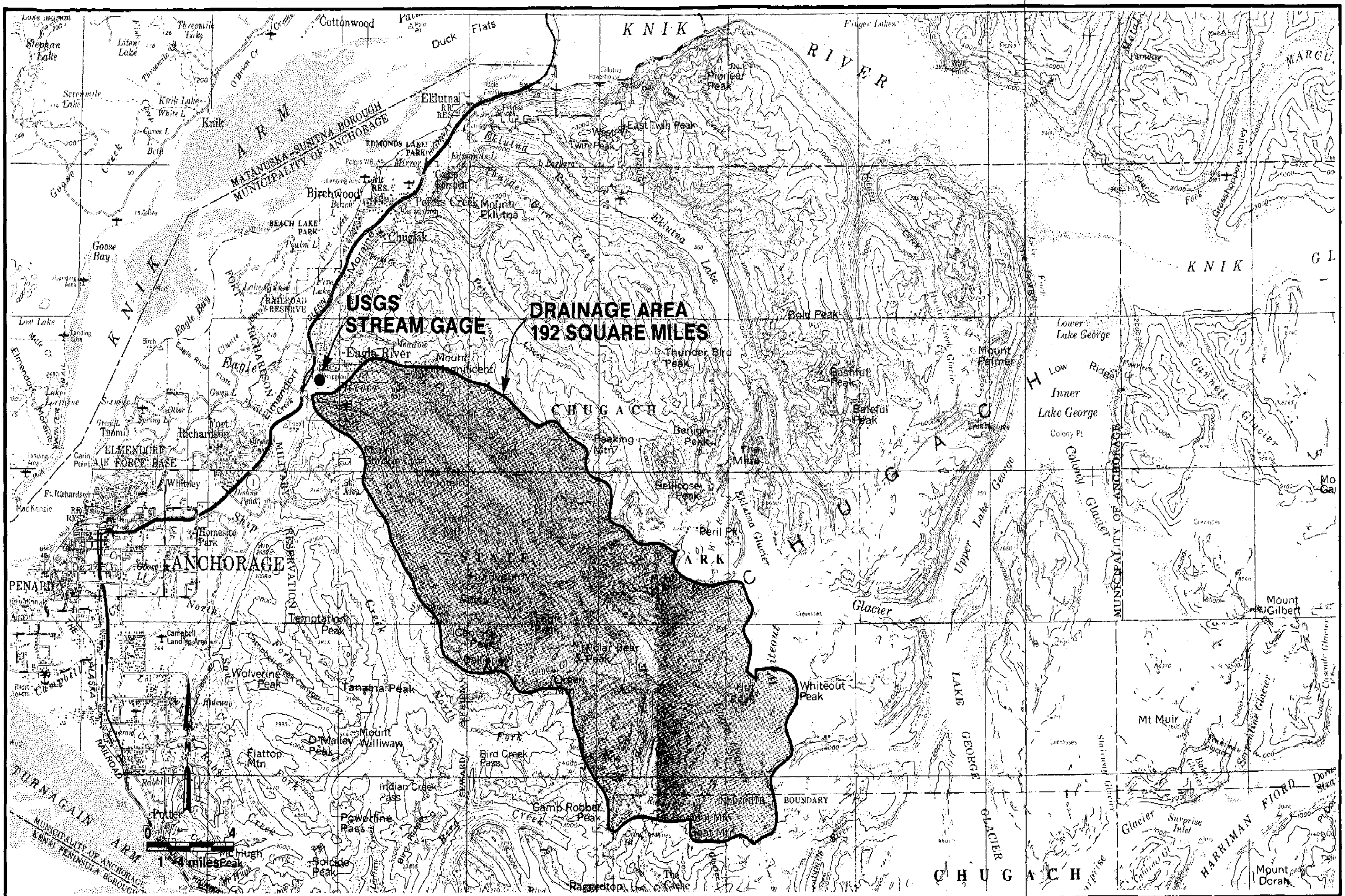
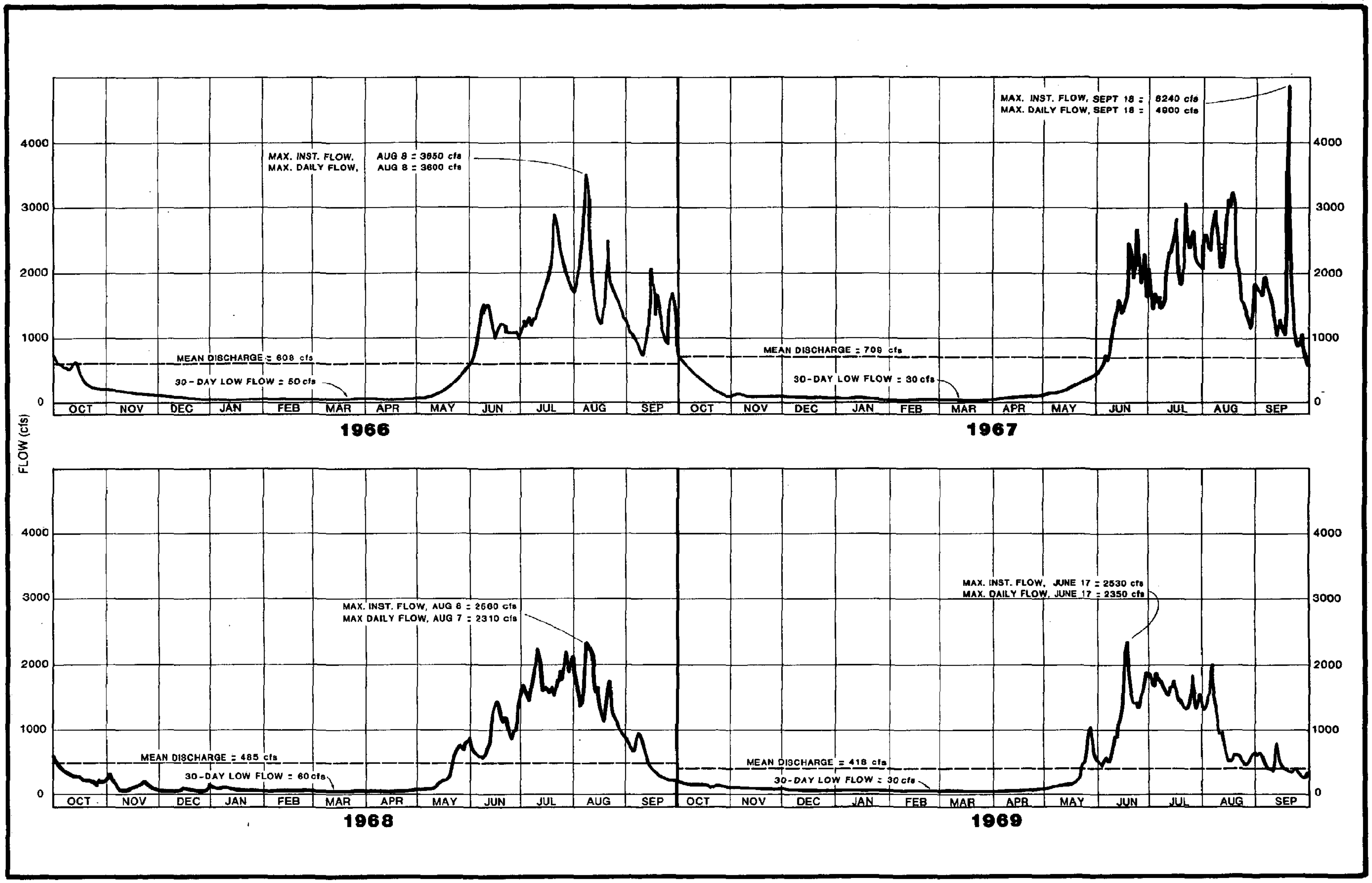
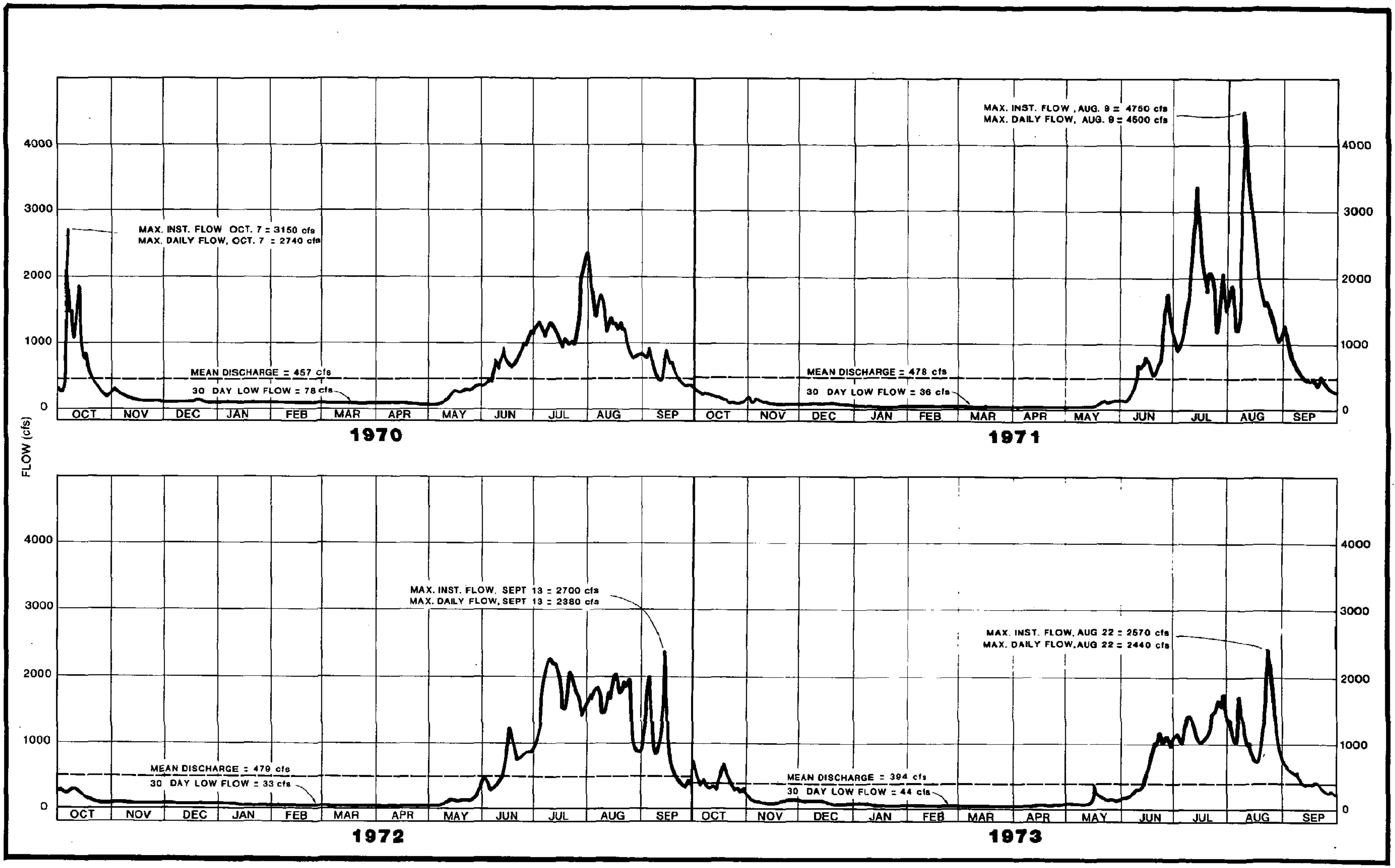


Figure 3-1
Eagle River
Drainage Basin



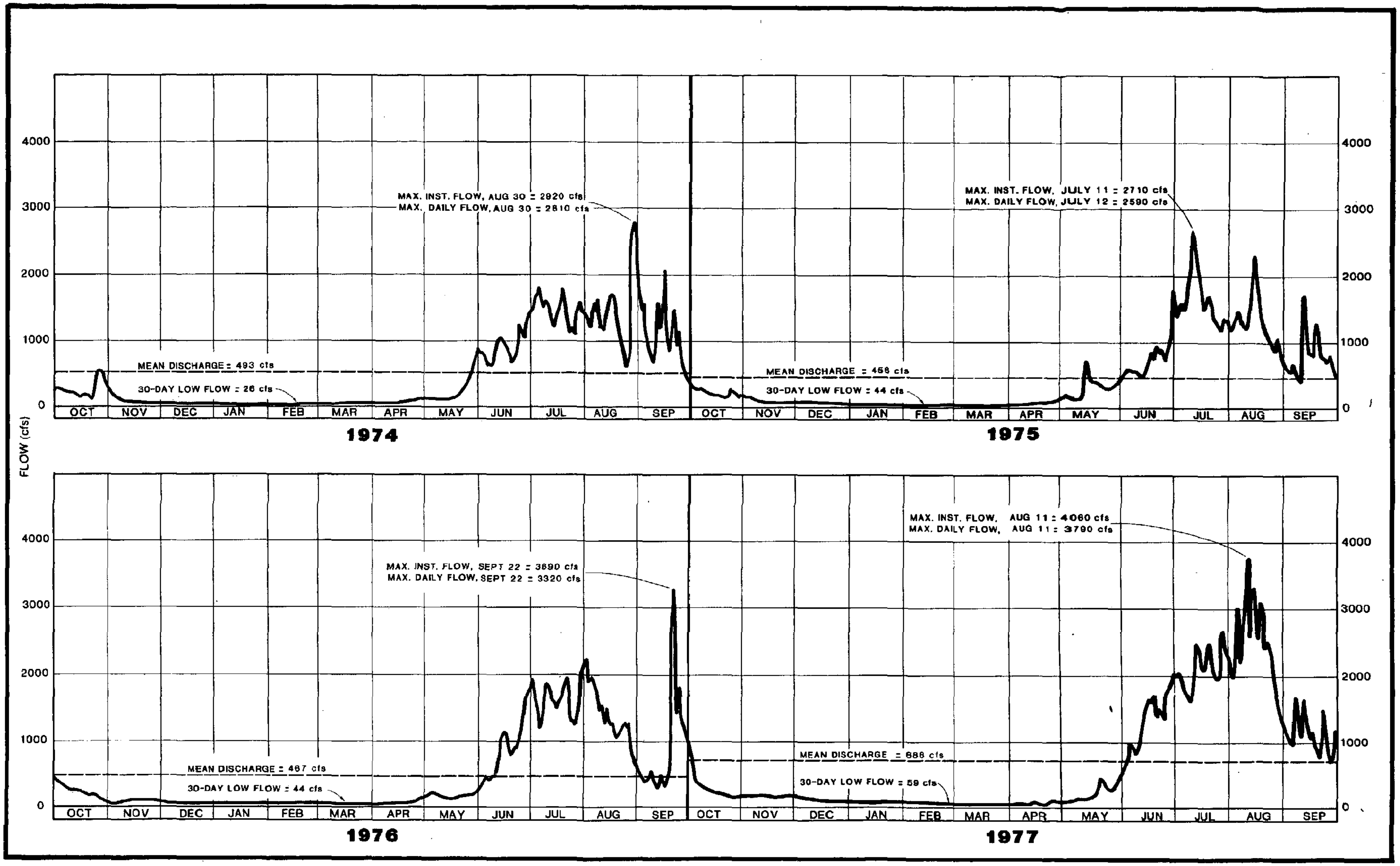
SOURCE: USGS Water Resources Data for Alaska

Figure 3-2
Eagle River at Eagle River Daily Discharge Hydrographs, 1966—1969



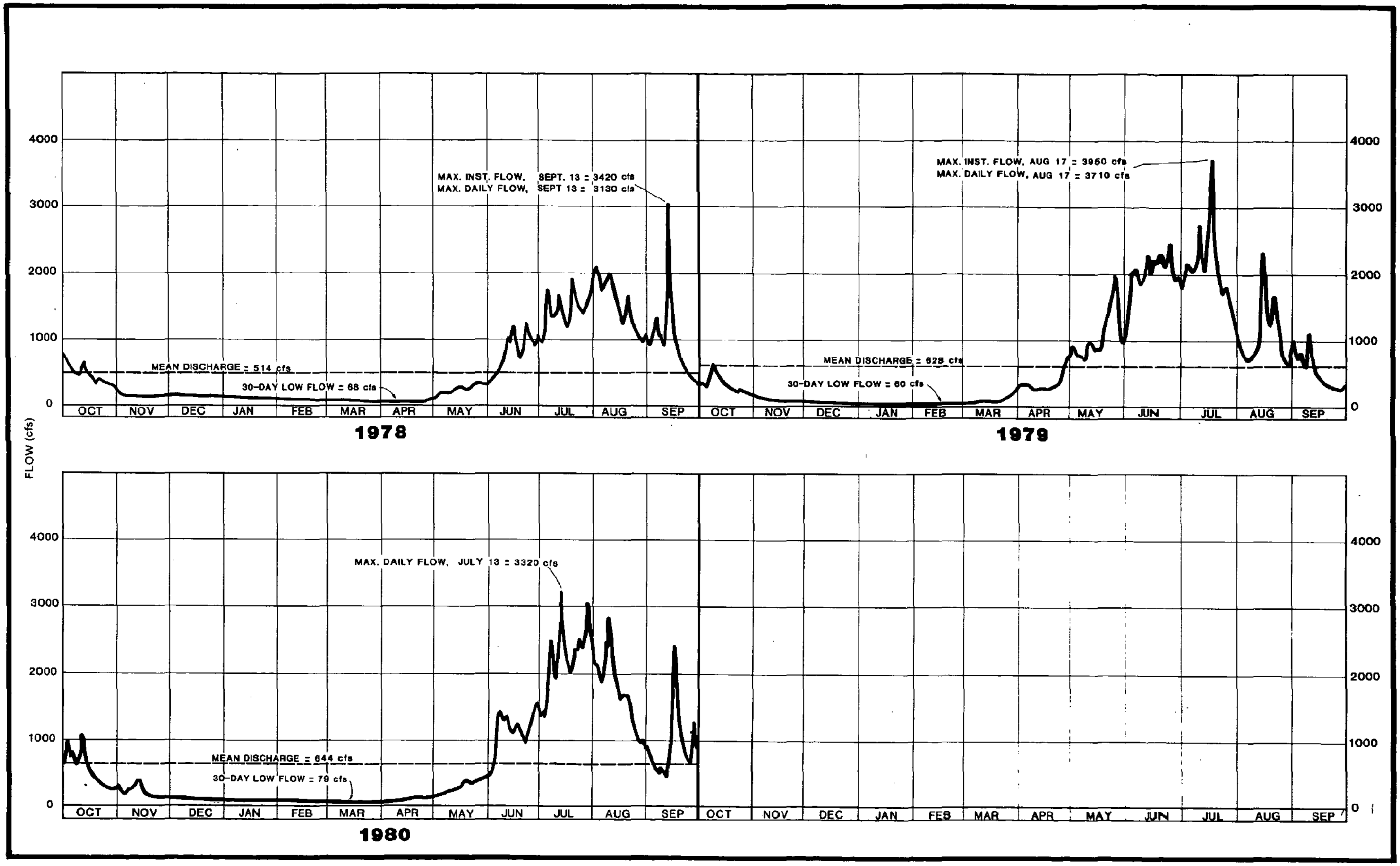
SOURCE: USGS Water Resources Data for Alaska

Figure 3-3
Eagle River at Eagle River
Daily Discharge
Hydrographs, 1970-1973



SOURCE: USGS Water Resources Data for Alaska

Figure 3-4
Eagle River at Eagle River
Daily Discharge
Hydrographs, 1974-1977



SOURCE: USGS Water Resources Data for Alaska

Figure 3-5
Eagle River at Eagle River
Daily Discharge
Hydrographs, 1978-1980

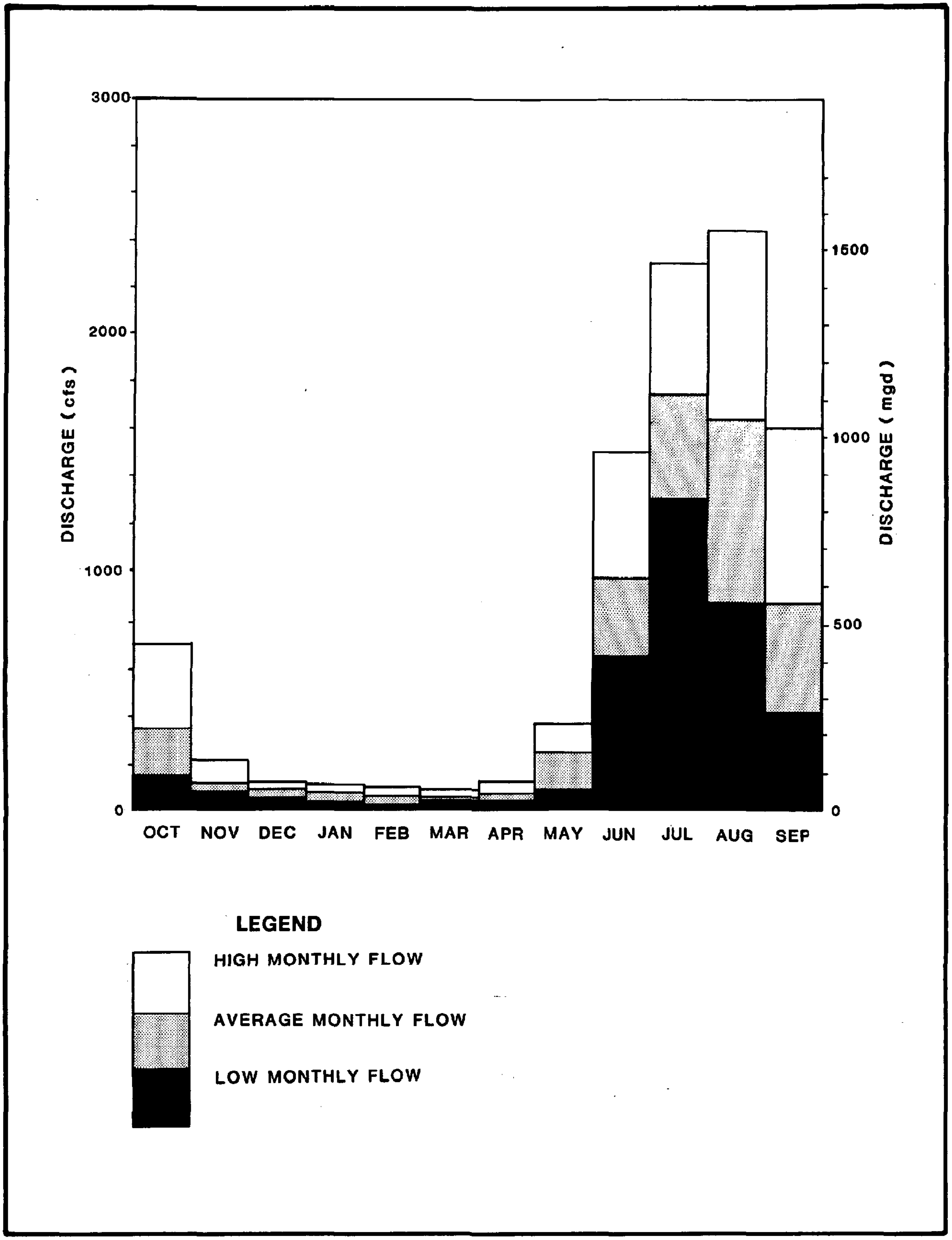


Figure 3-6
Low, Average, and High Monthly Flows

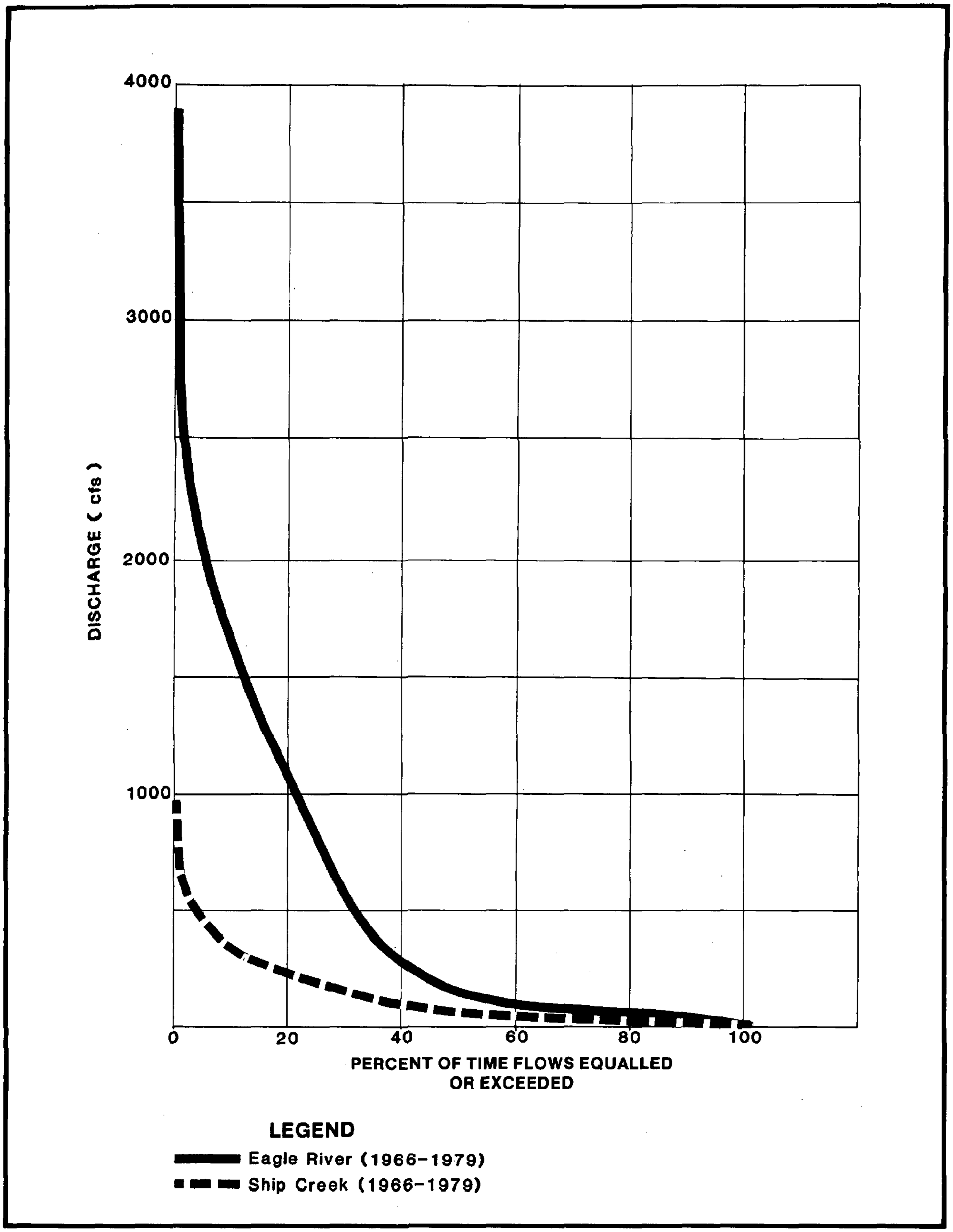


Figure 3-7
Daily Flow
Duration Curves

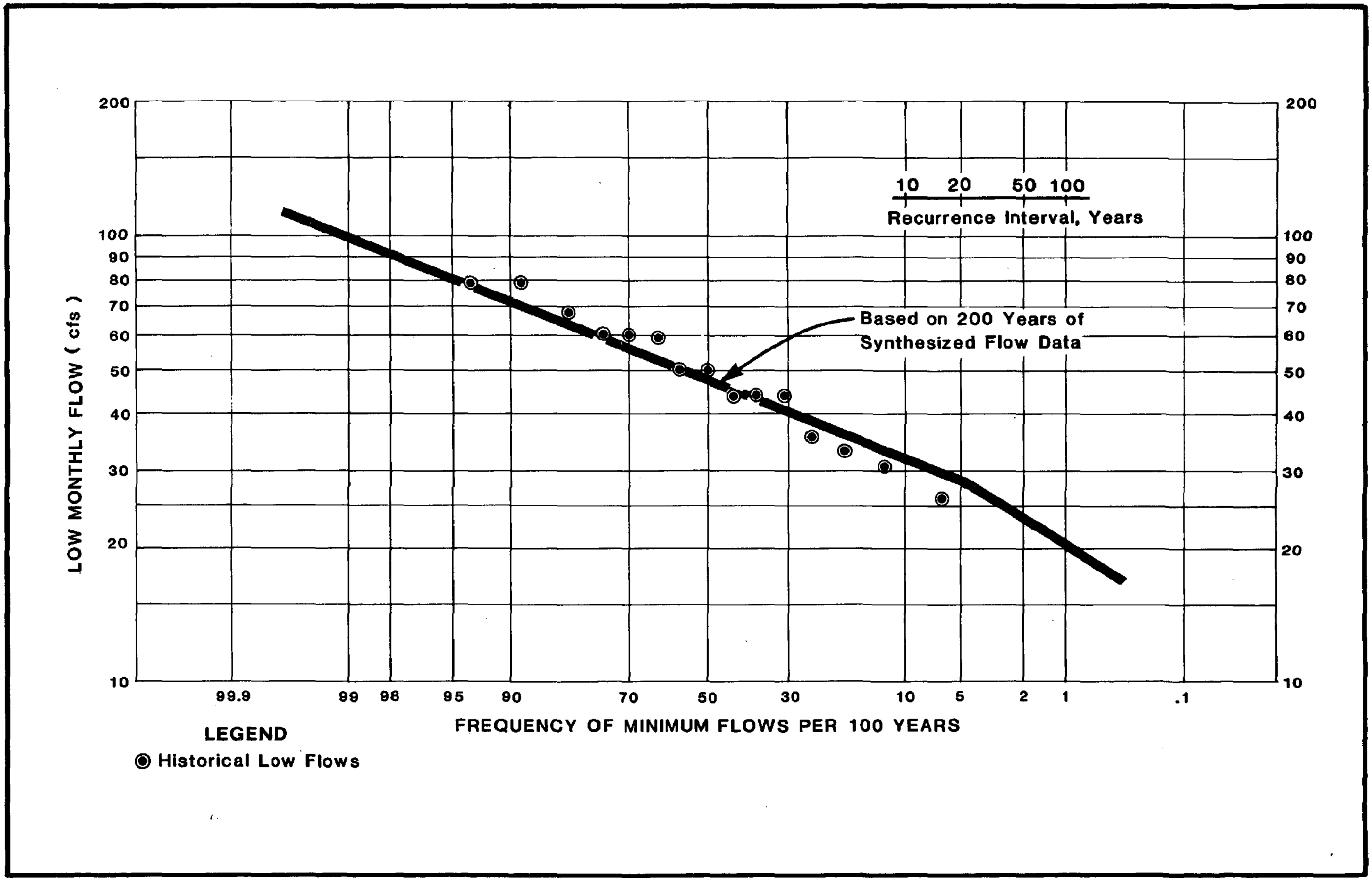


Figure 3-8
Low-Flow Frequency Curve

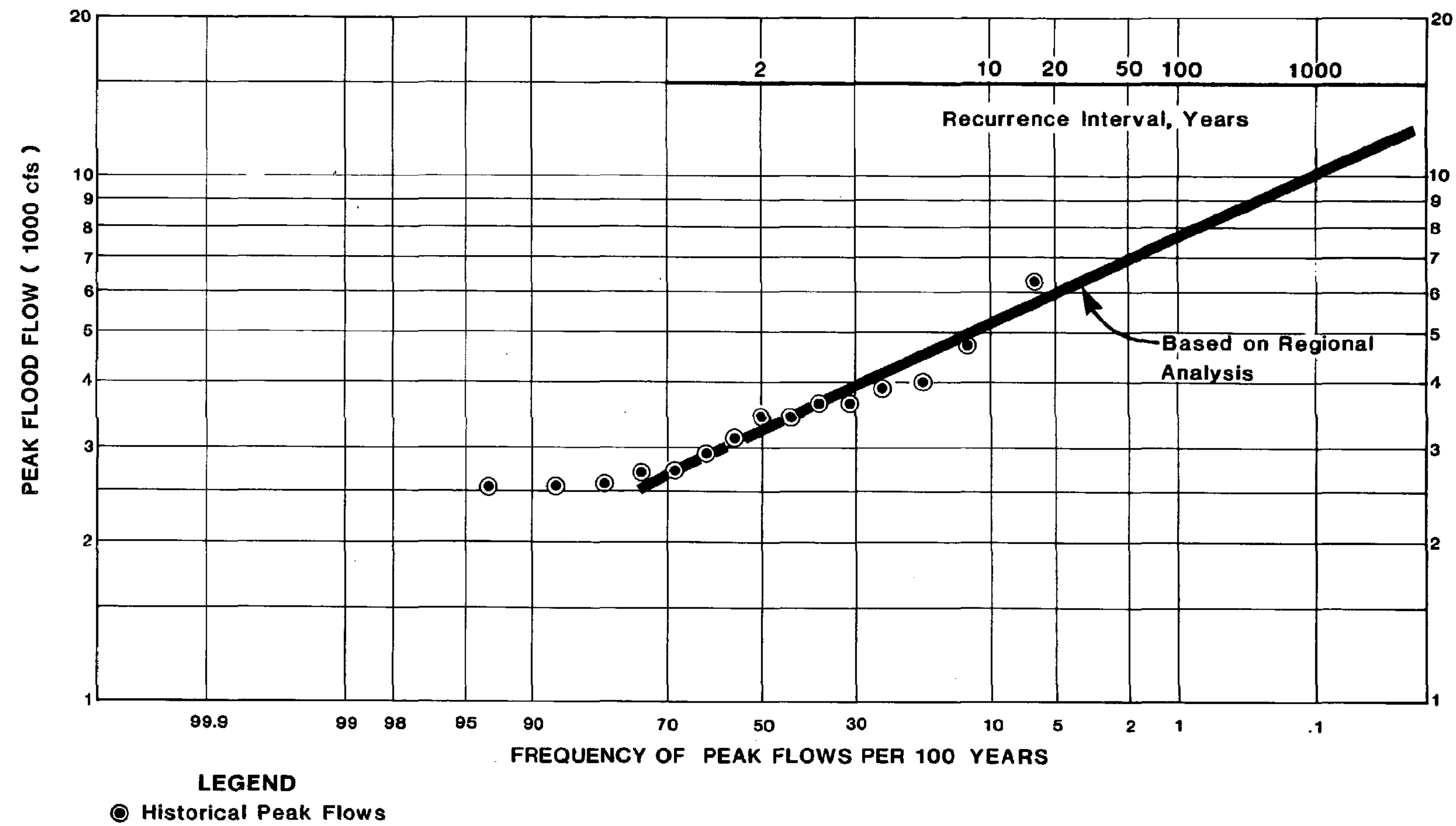


Figure 3-9
Flood Frequency Curve

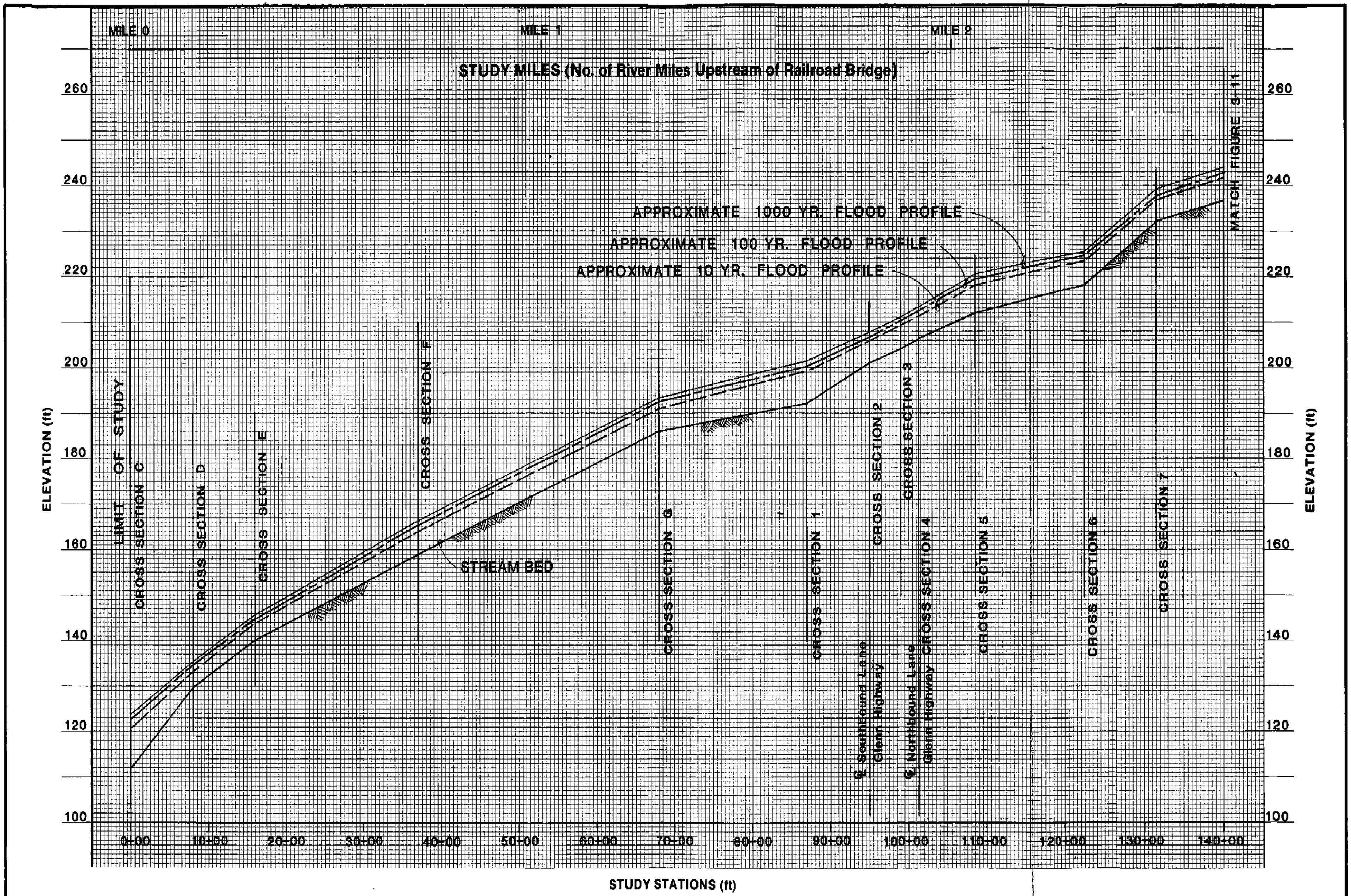


Figure 3-10
Approximate Flood Profiles
Study Mile 0 to 2.7

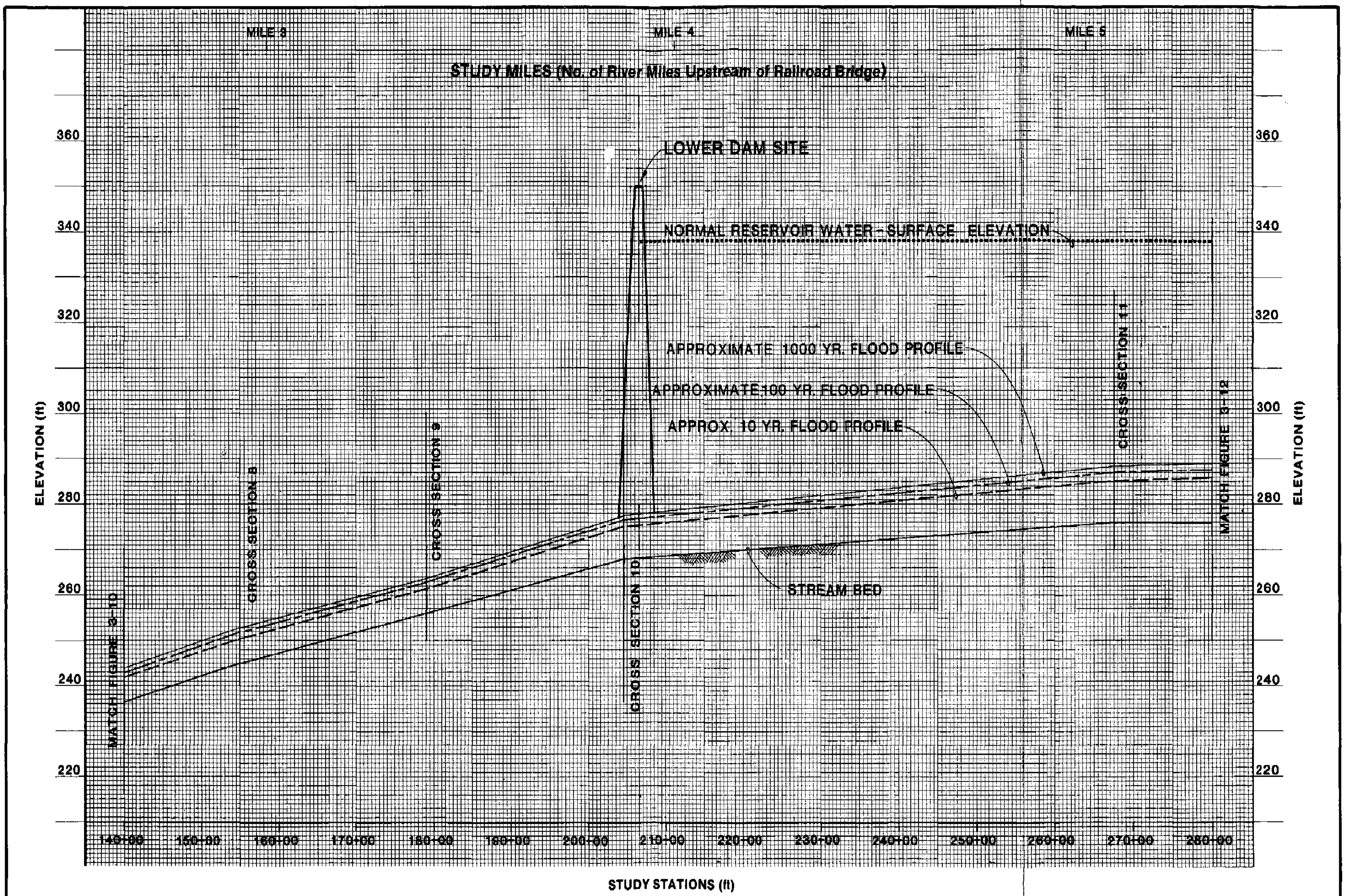


Figure 3-11
Approximate Flood Profiles
Study Mile 2.7 to 5.3

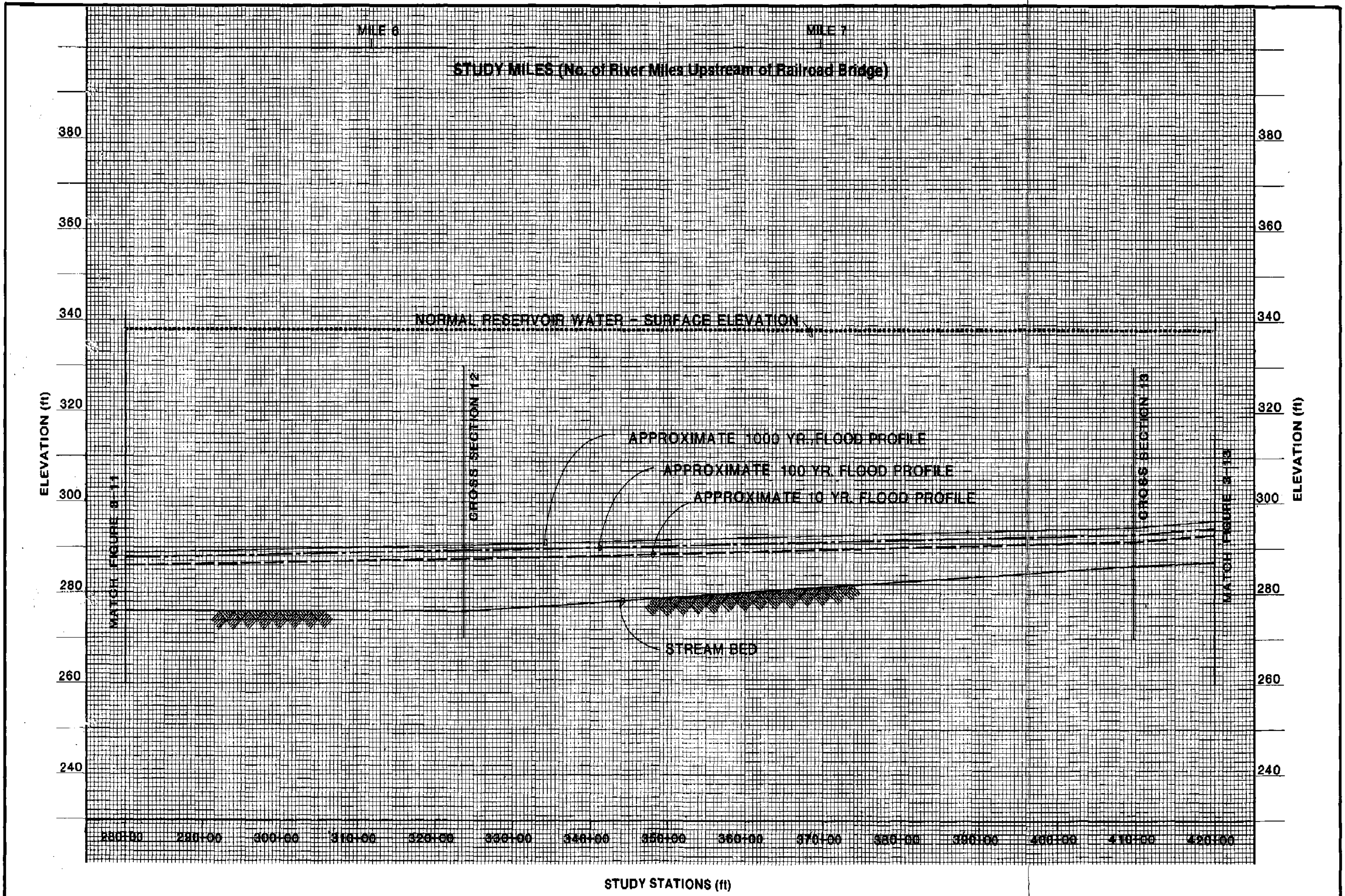


Figure 3-12
Approximate Flood Profiles
Study Mile 5.3 to 8.0

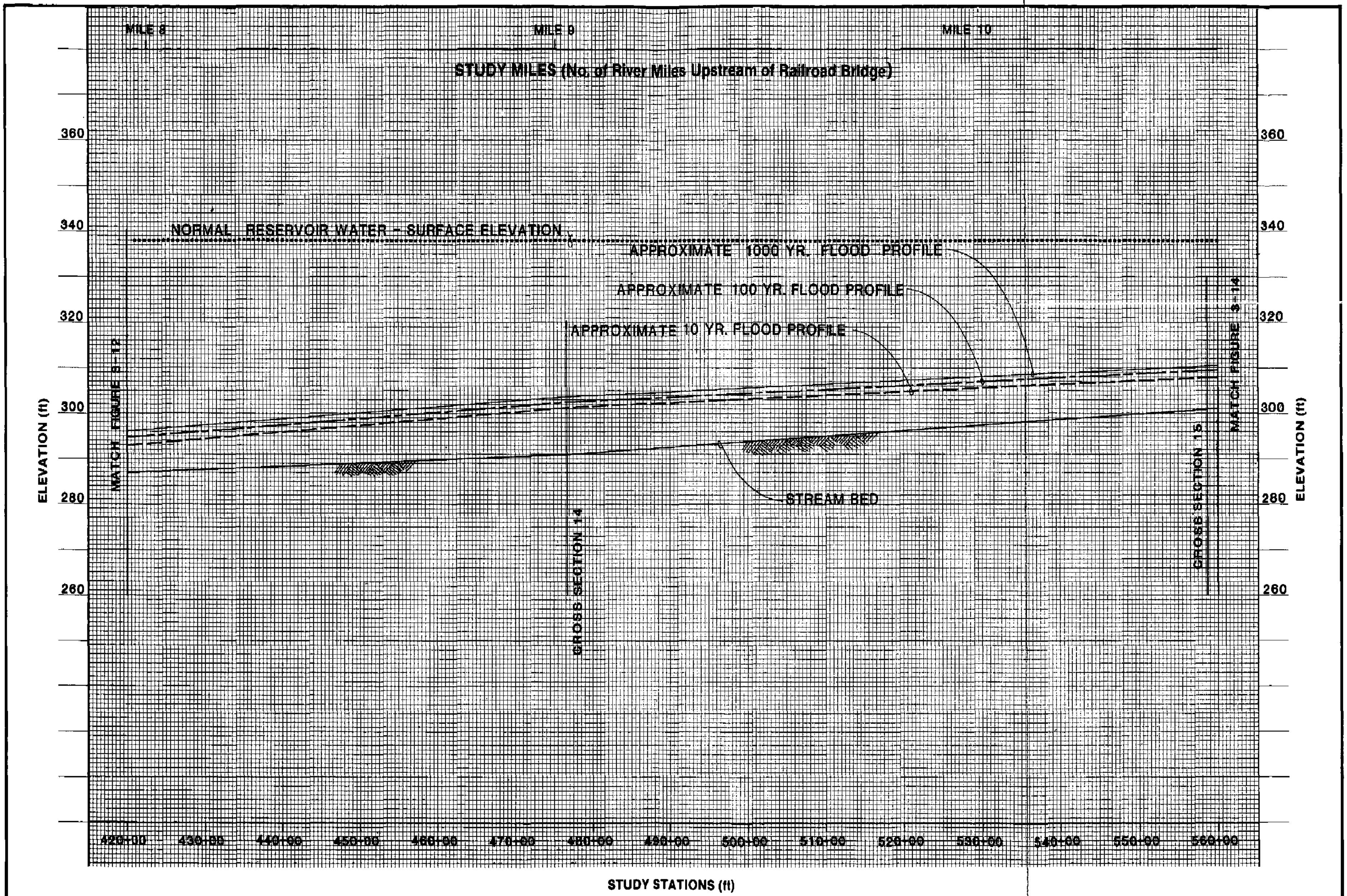


Figure 3-13
Approximate Flood Profiles
Study Mile 8.0 to 10.6

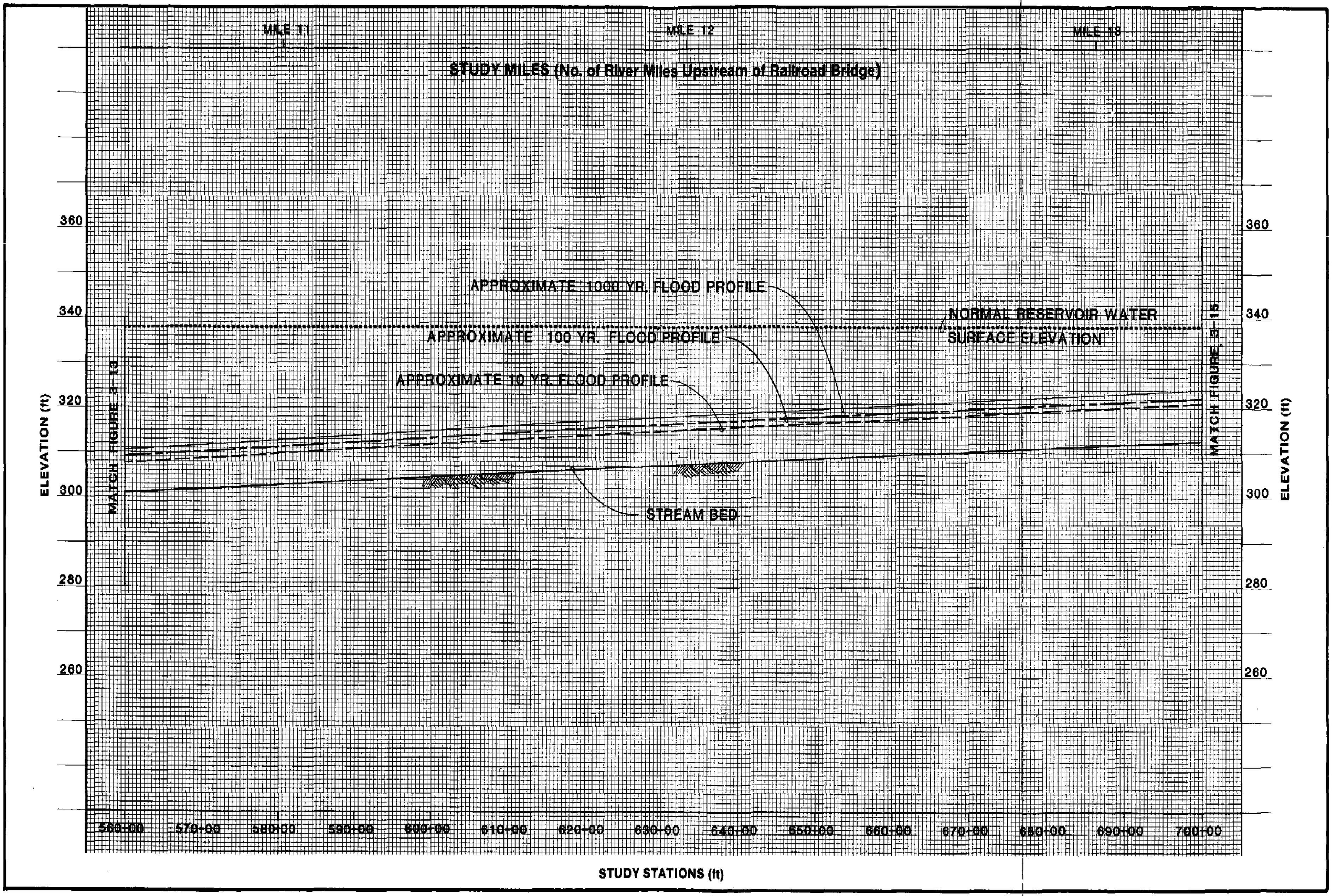


Figure 3-14
Approximate Flood Profiles
Study Mile 10.6 to 13.3

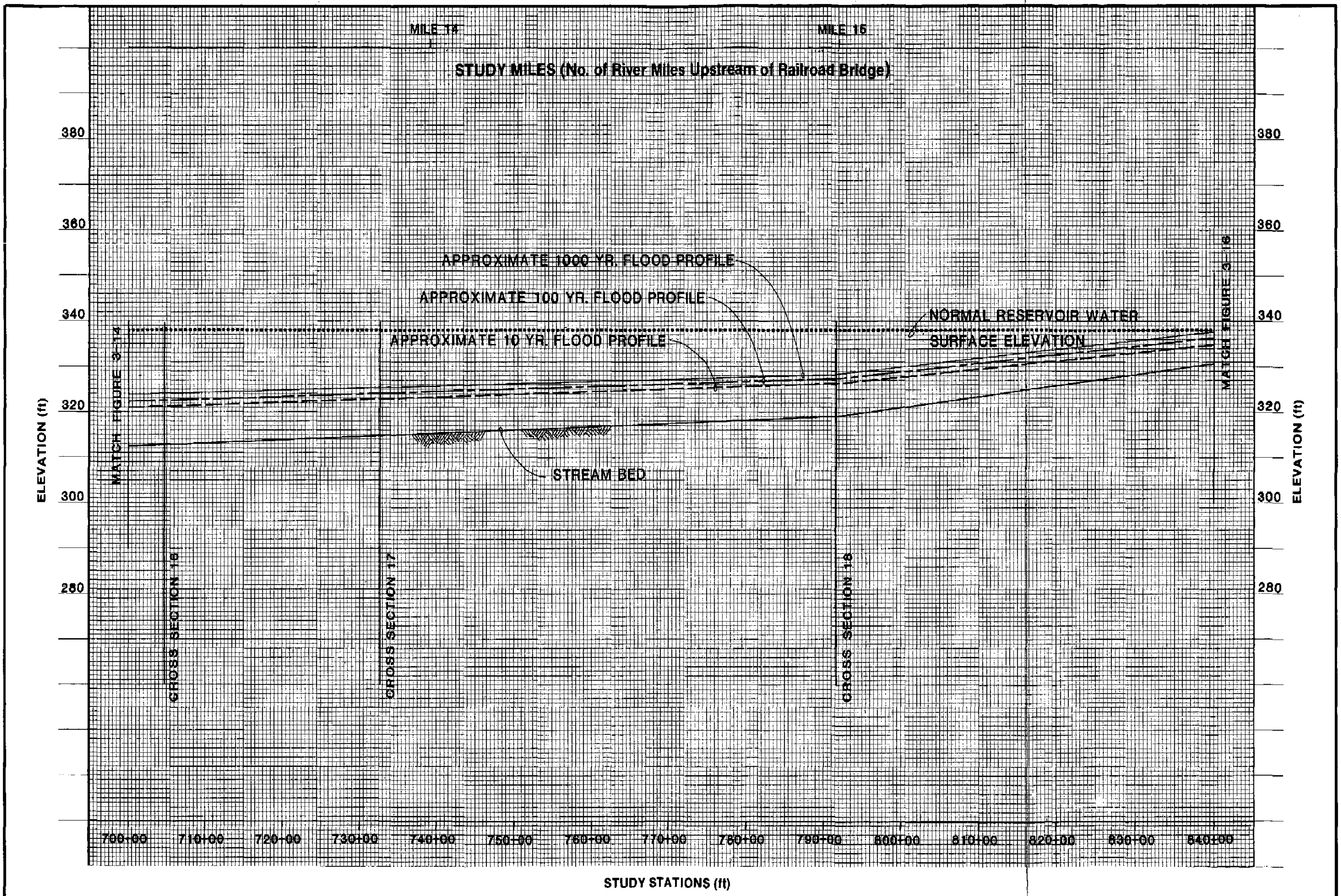


Figure 3-15
Approximate Flood Profiles
Study Mile 13.3 to 15.9

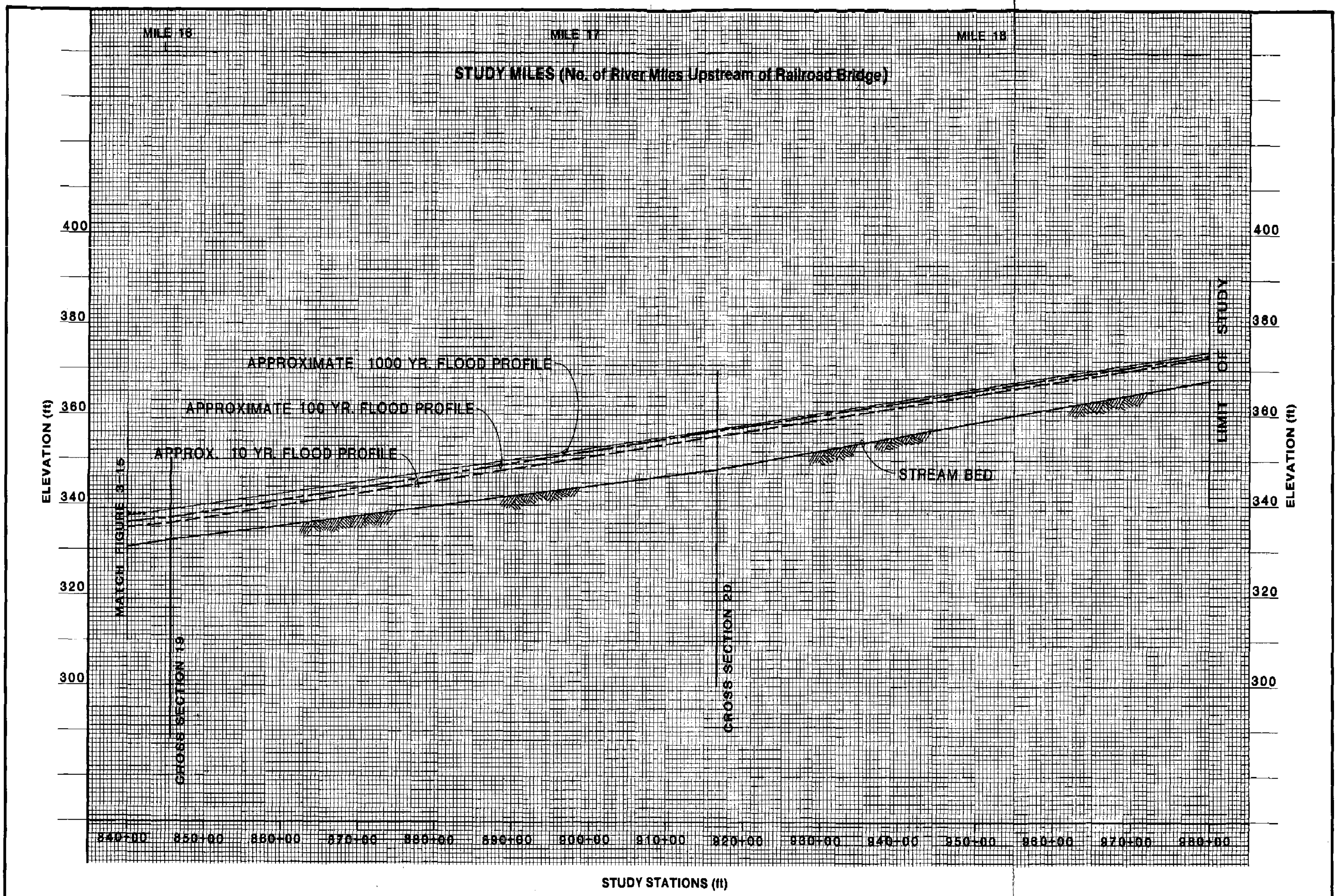


Figure 3-16
Approximate Flood Profiles
Study Mile 15.9 to 18.6

NOTE: Where Flood Boundaries Overlap, Only the Approx. 100 Yr. Boundary Has Been Shown

Figure 3-17
Approximate Flood Plain Boundaries
Study Mile 0 to 4.1

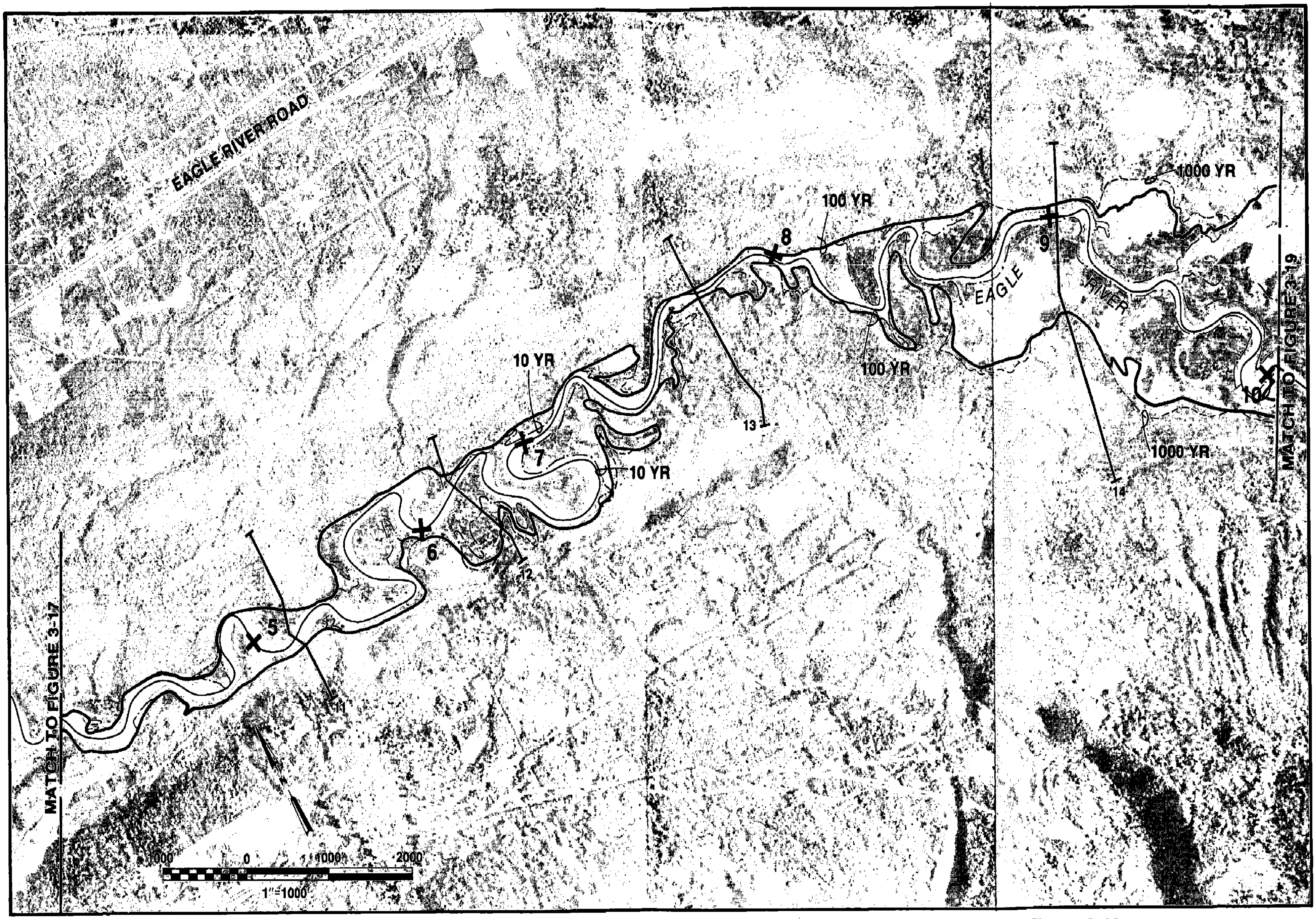


Figure 3-18
Approximate Flood Plain Boundaries
Study Mile 4.1 to 10

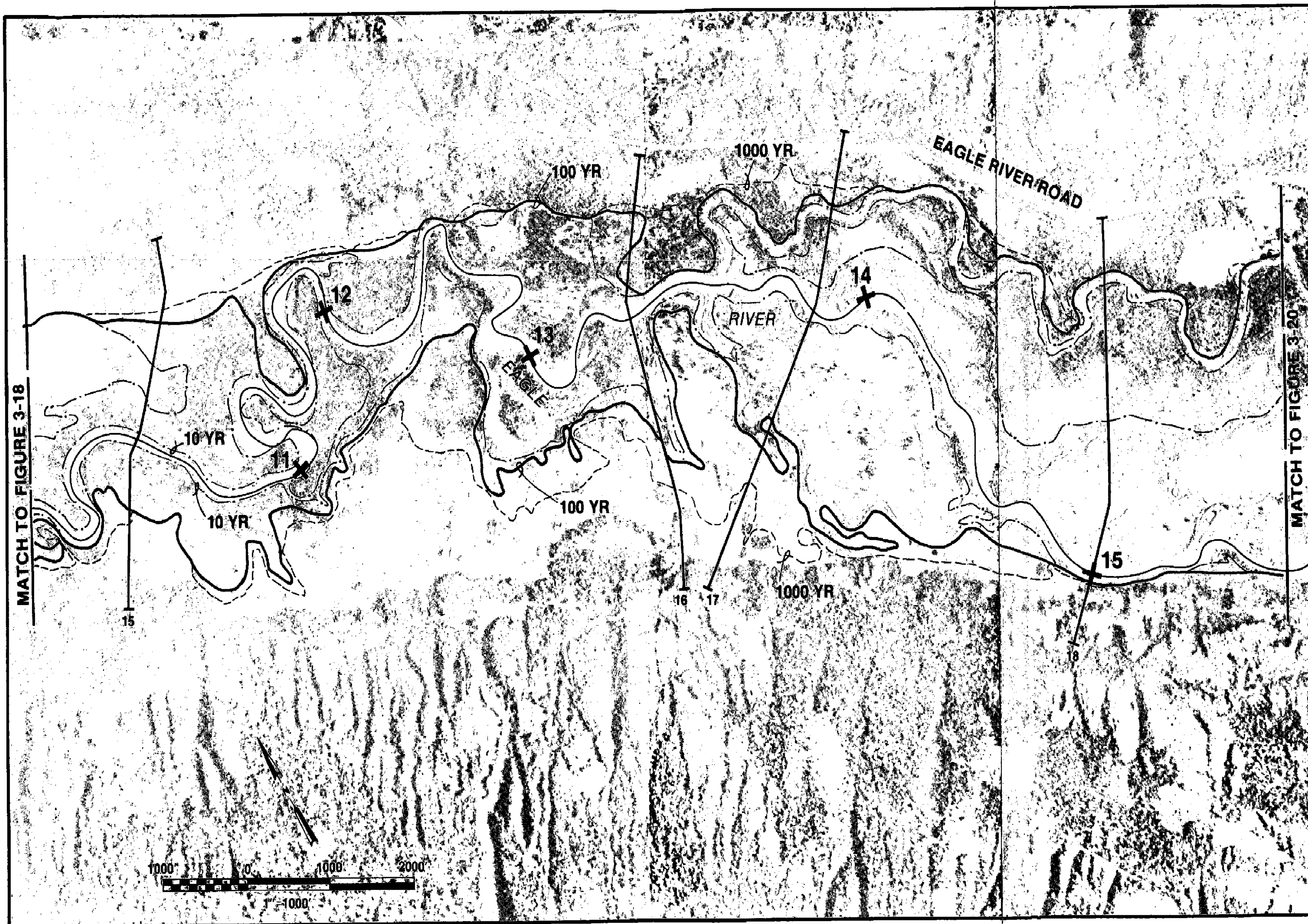


Figure 3-19
Approximate Flood Plain Boundaries
Study Mile 10 to 15.5

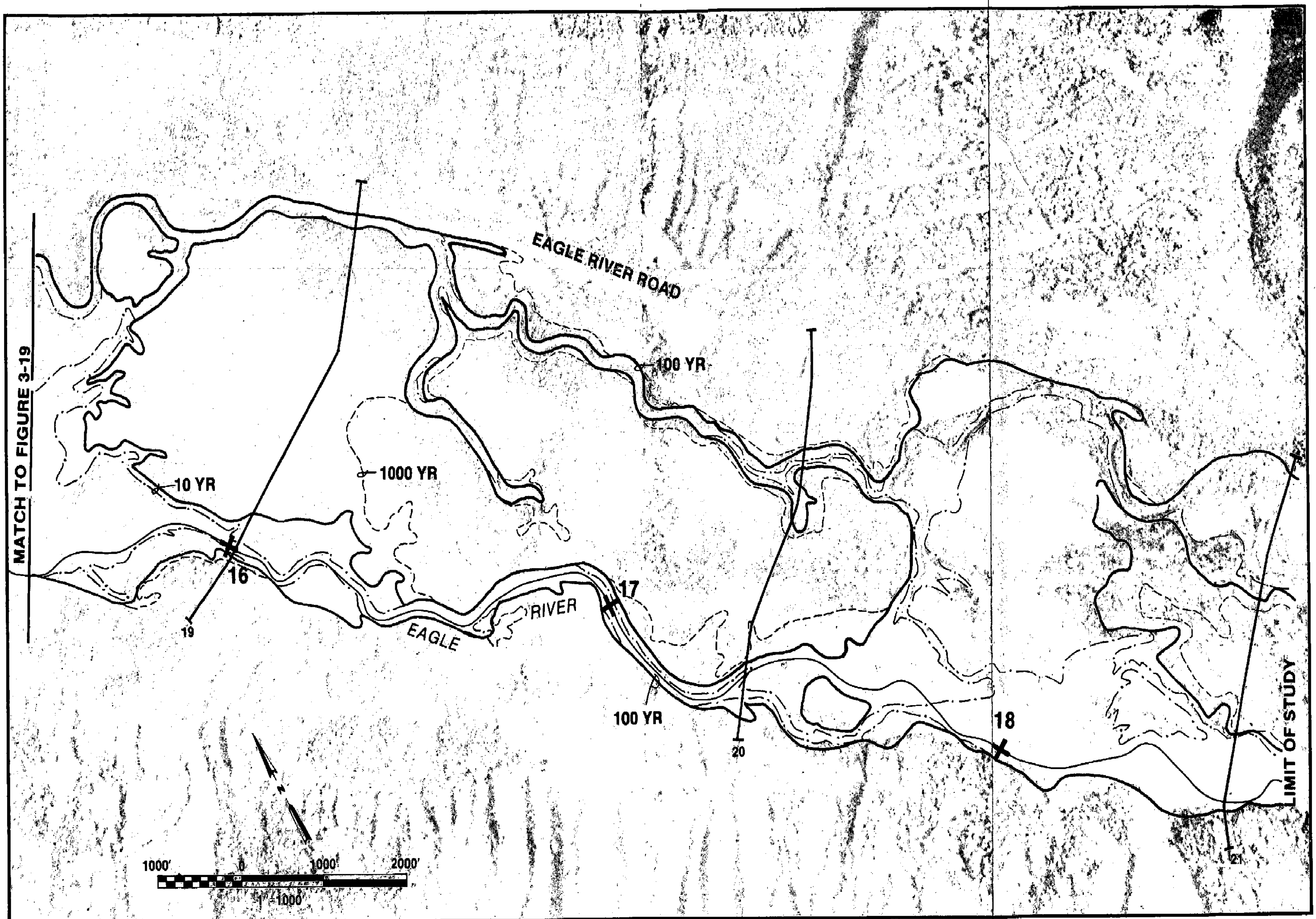


Figure 3-20
Approximate Flood Plain Boundaries
Study Mile 15.5 to 18.7

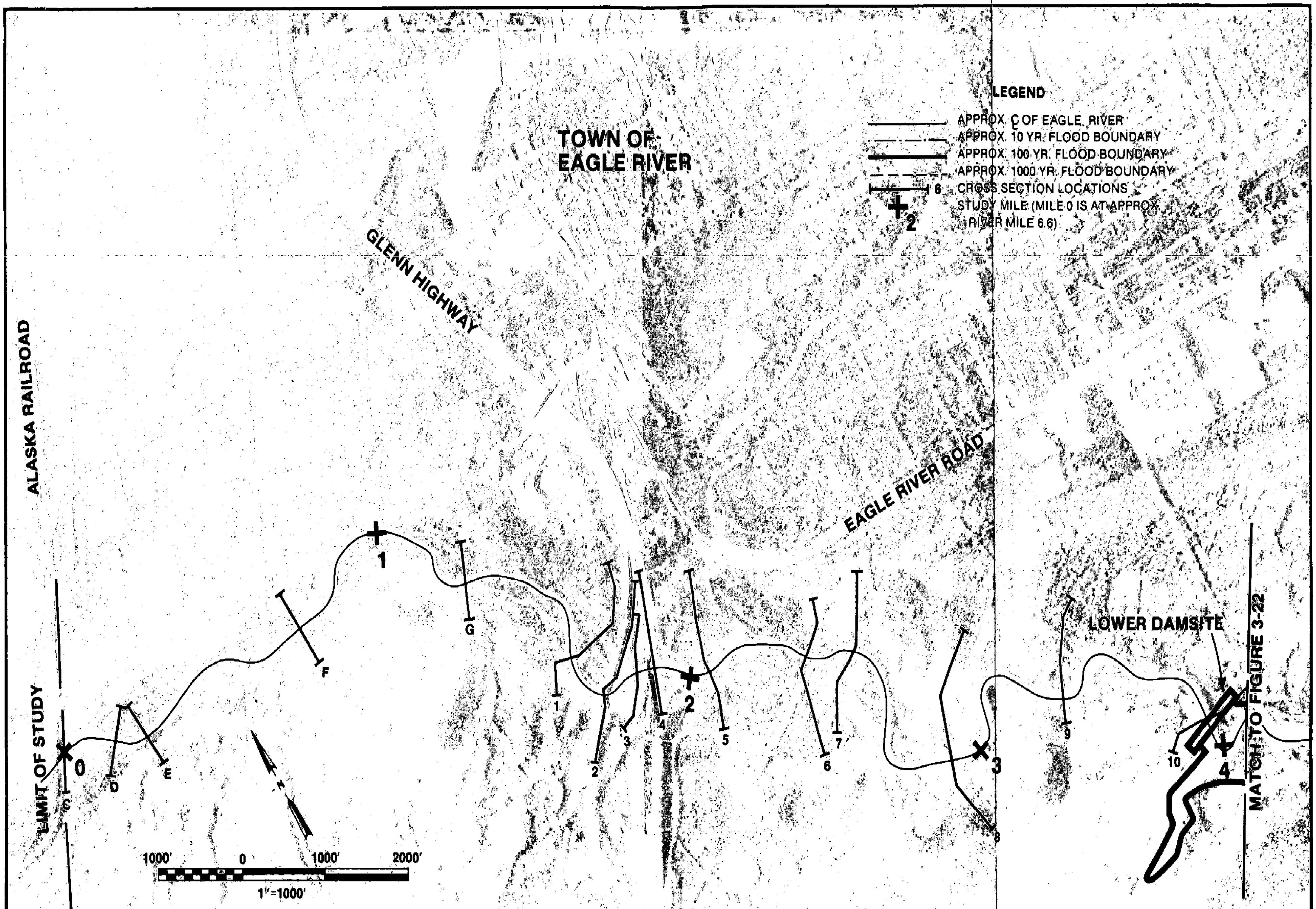


Figure 3-21
Approximate Reservoir Limits
Study Mile 0 to 4.1

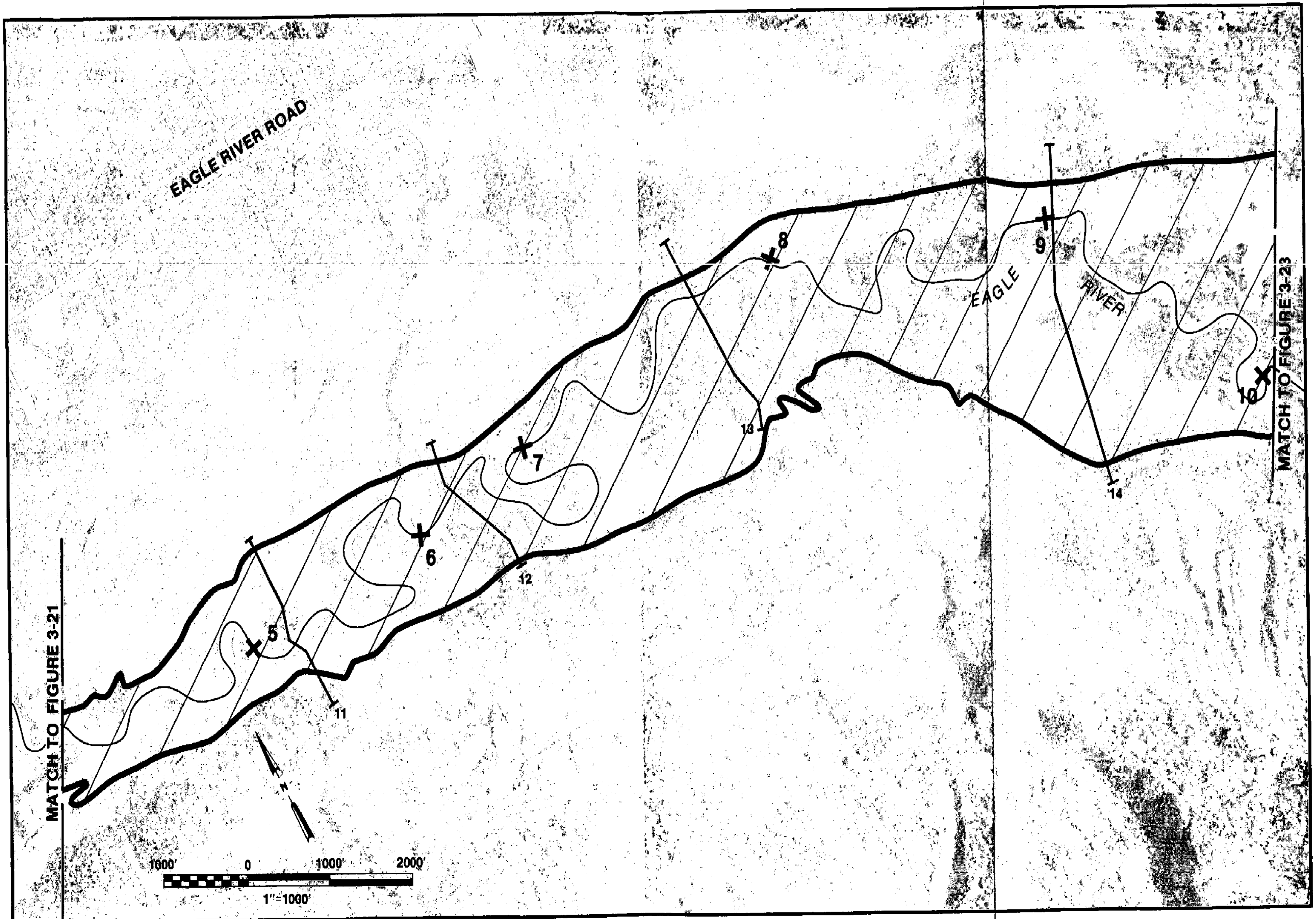


Figure 3-22
Approximate Reservoir Limits
Study Mile 4.1 to 10

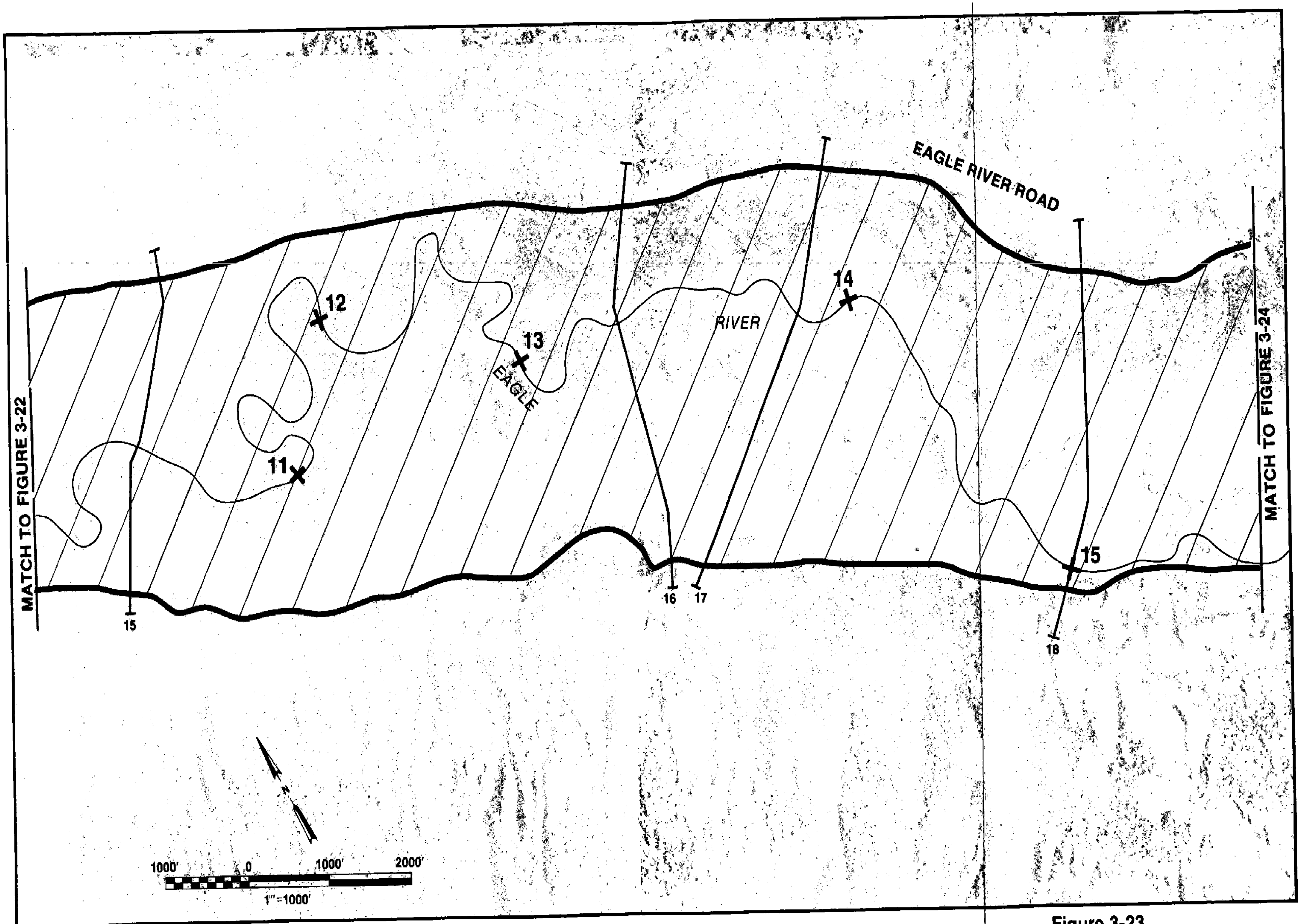


Figure 3-23
Approximate Reservoir Limits
Study Mile 10 to 15.5

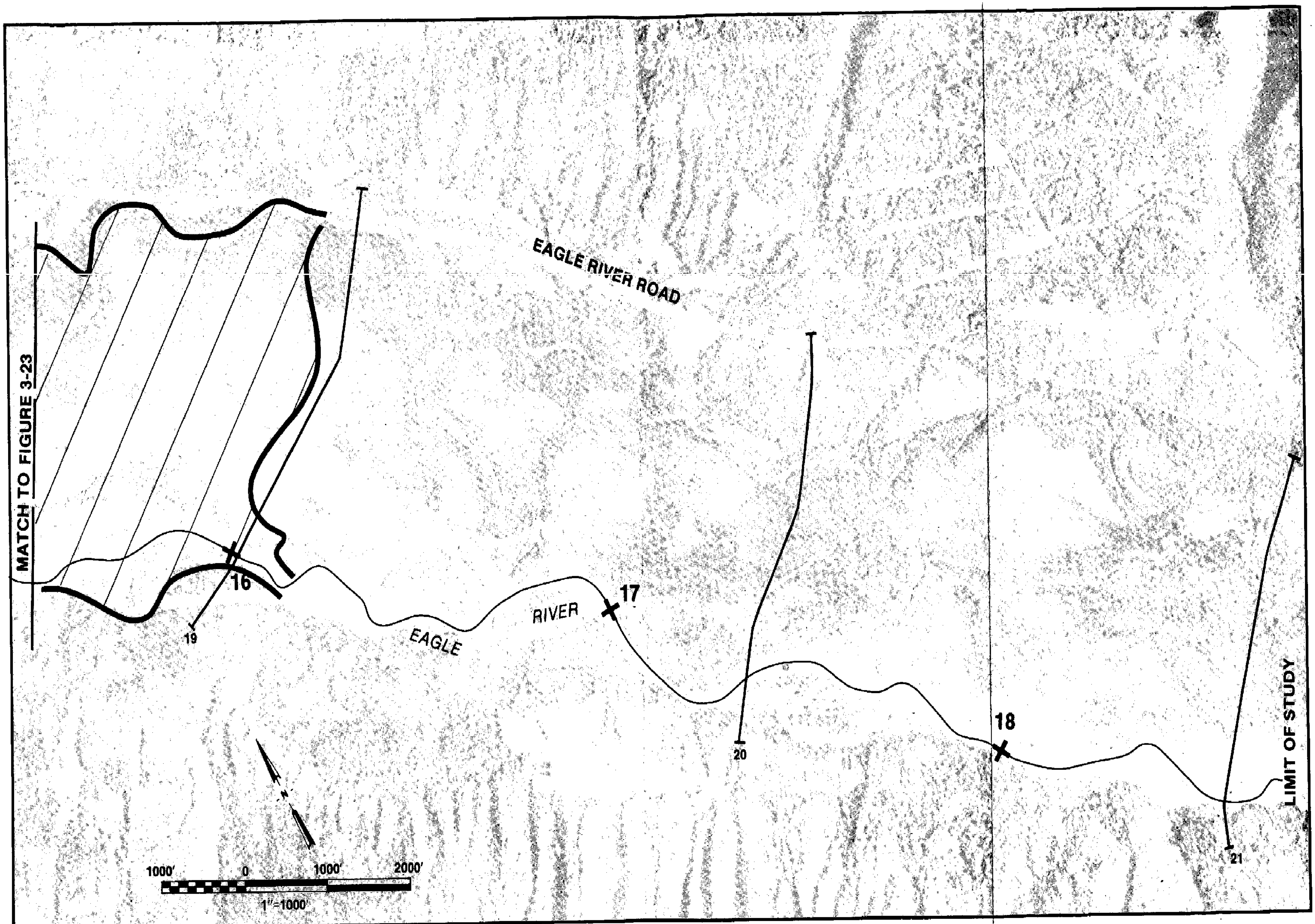


Figure 3-24
Approximate Reservoir Limits
Study Mile 15.5 to 18.7

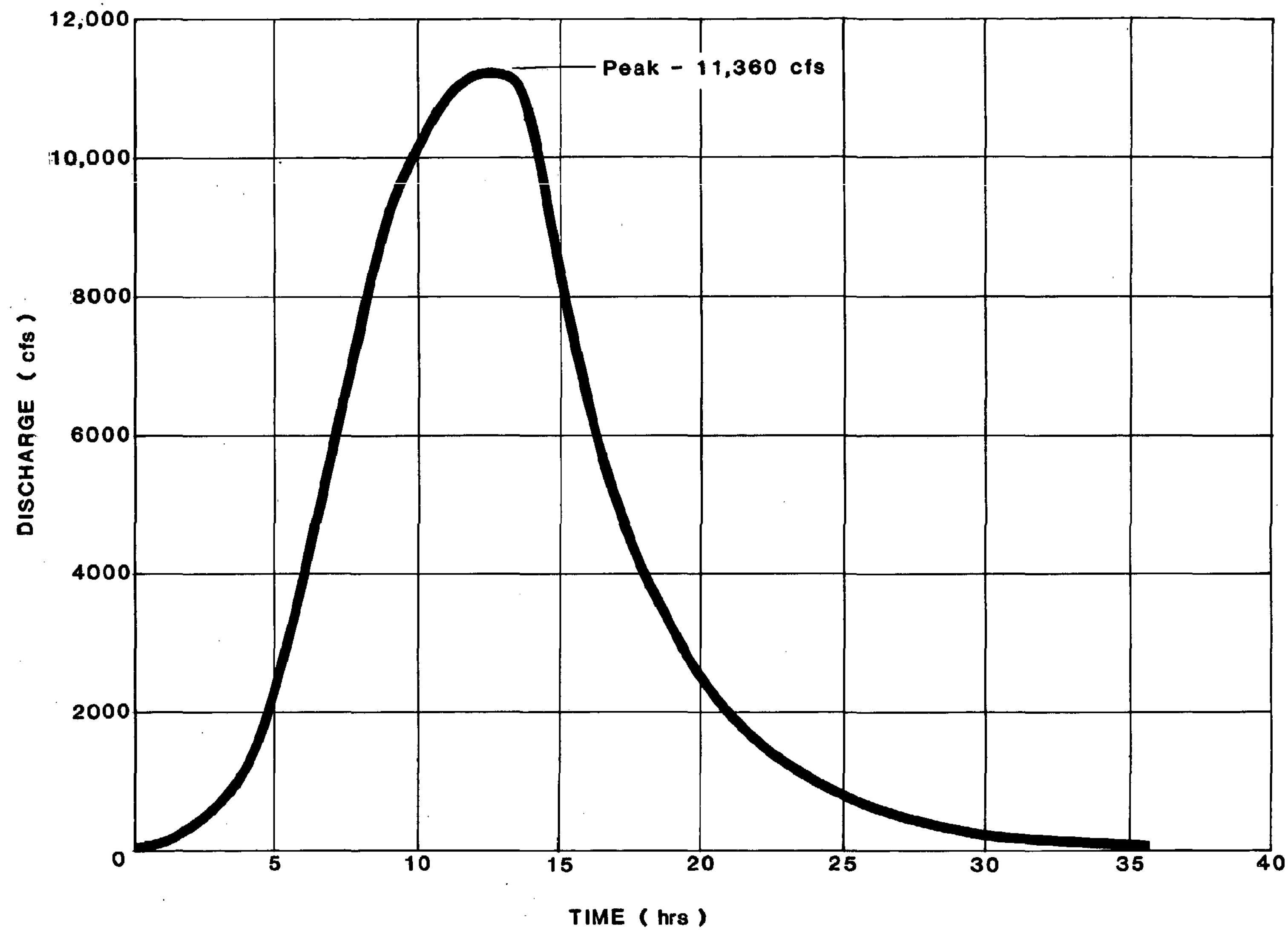


Figure 3-25
One-Hour Unit Hydrograph

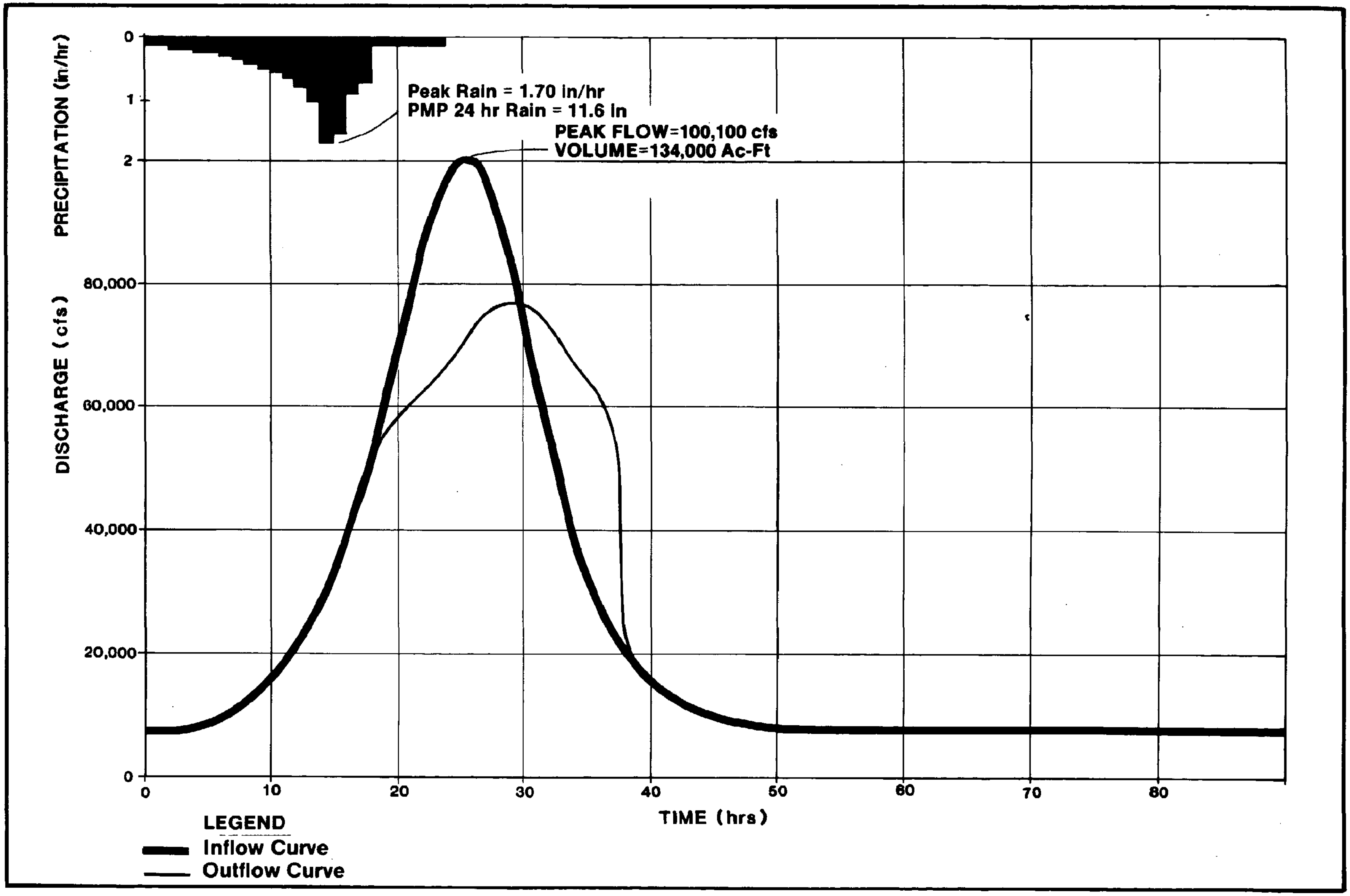
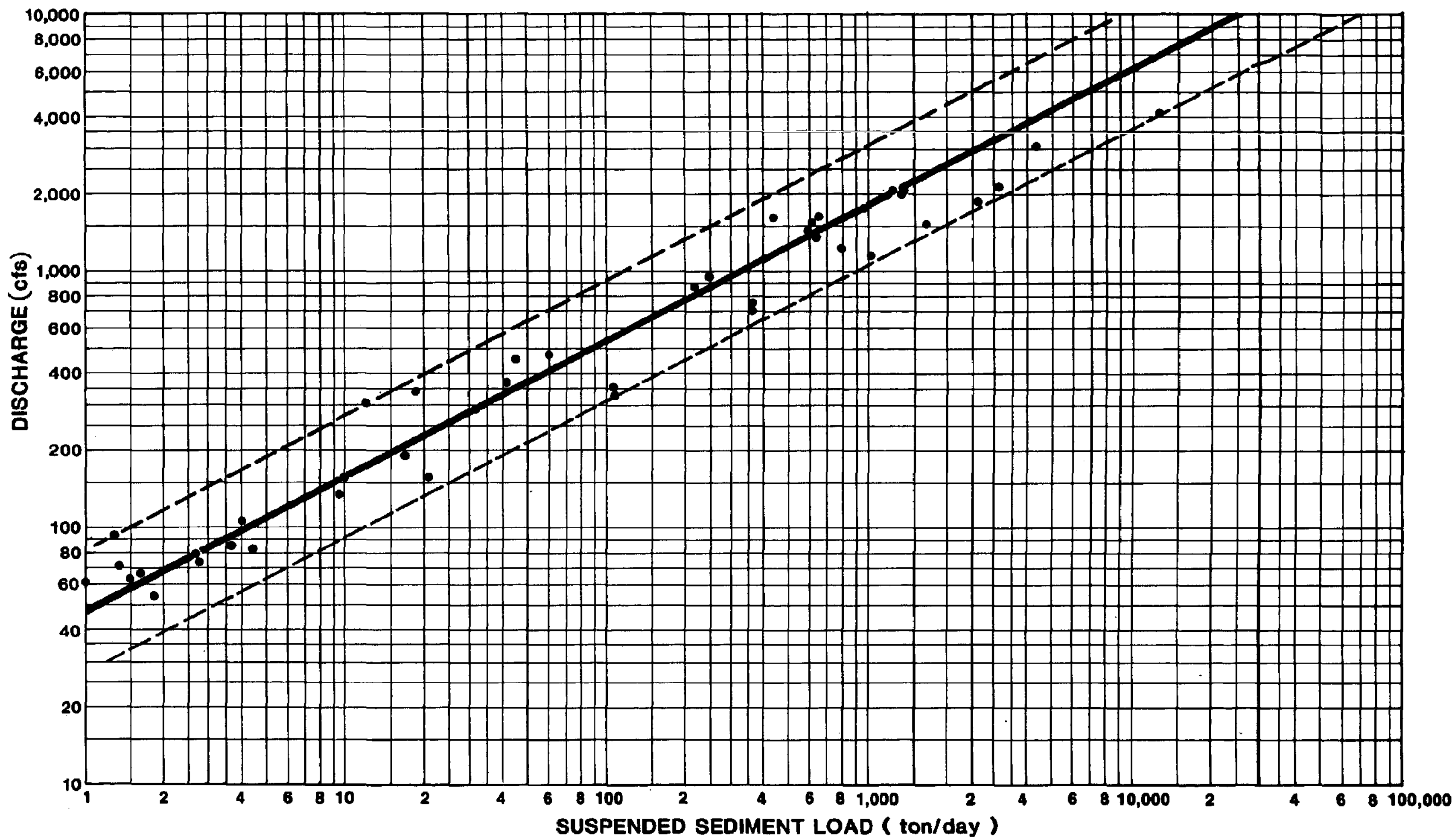
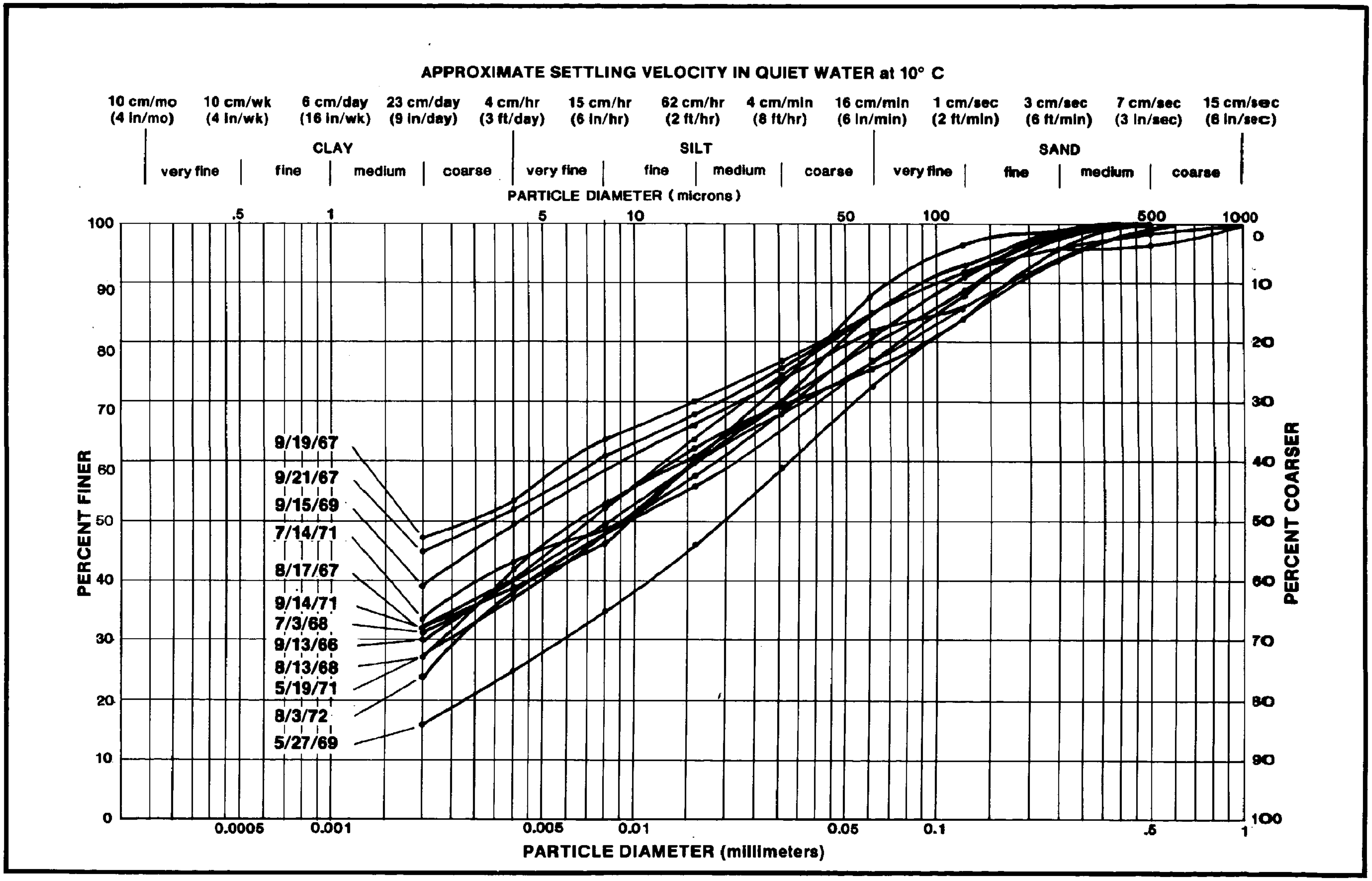


Figure 3-26
PMF Inflow and Routed Outflow



DATA SOURCE: U.S. Army Corps of Engineers. 1979.

Figure 3-27
Eagle River at Eagle River
Suspended Sediment Load
1966-1972



DATA SOURCE: U.S. Army Corps of Engineers. 1979.

Figure 3-28
Eagle River at Eagle River
Sediment Size Analysis
1966-1972

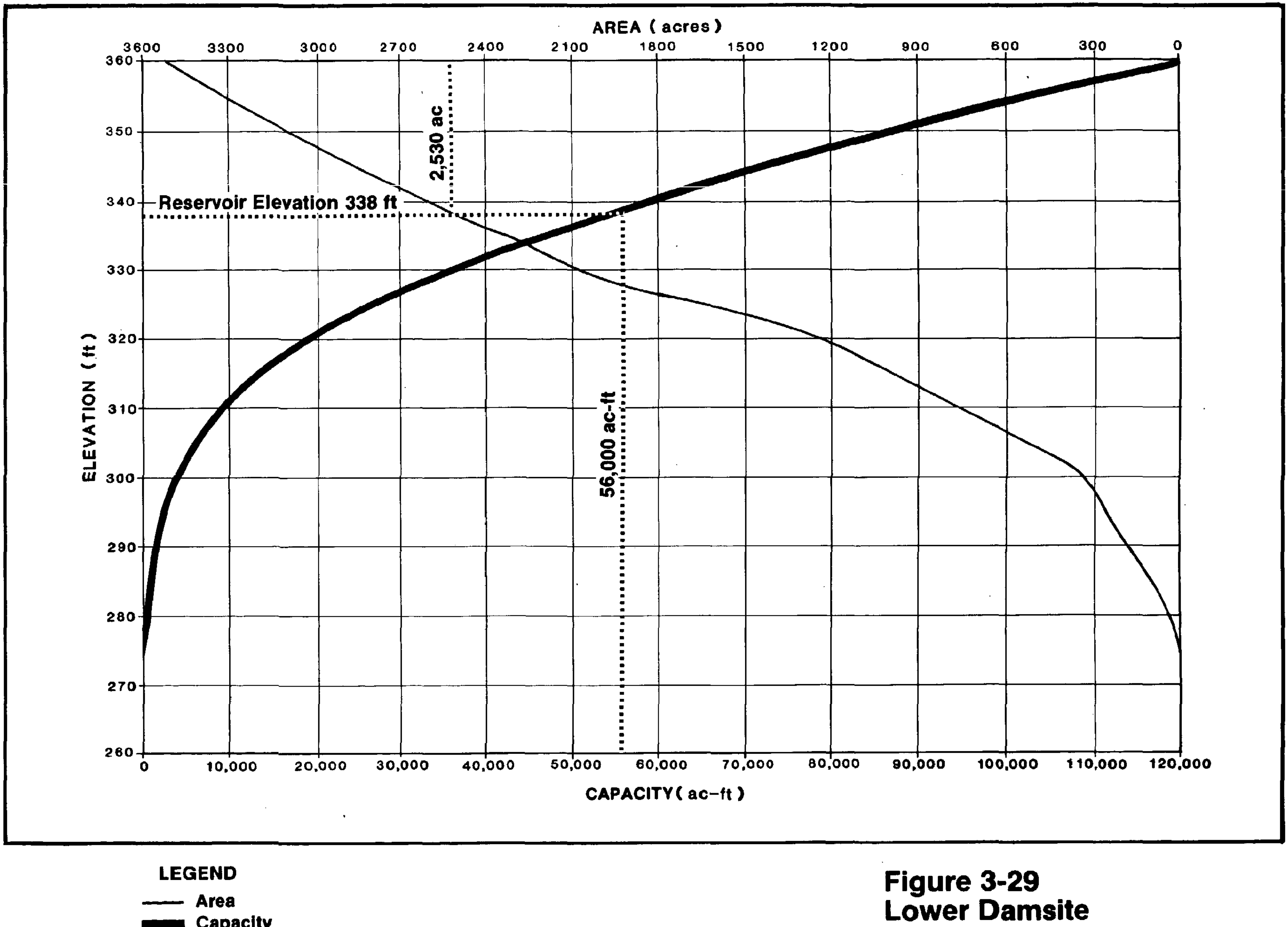
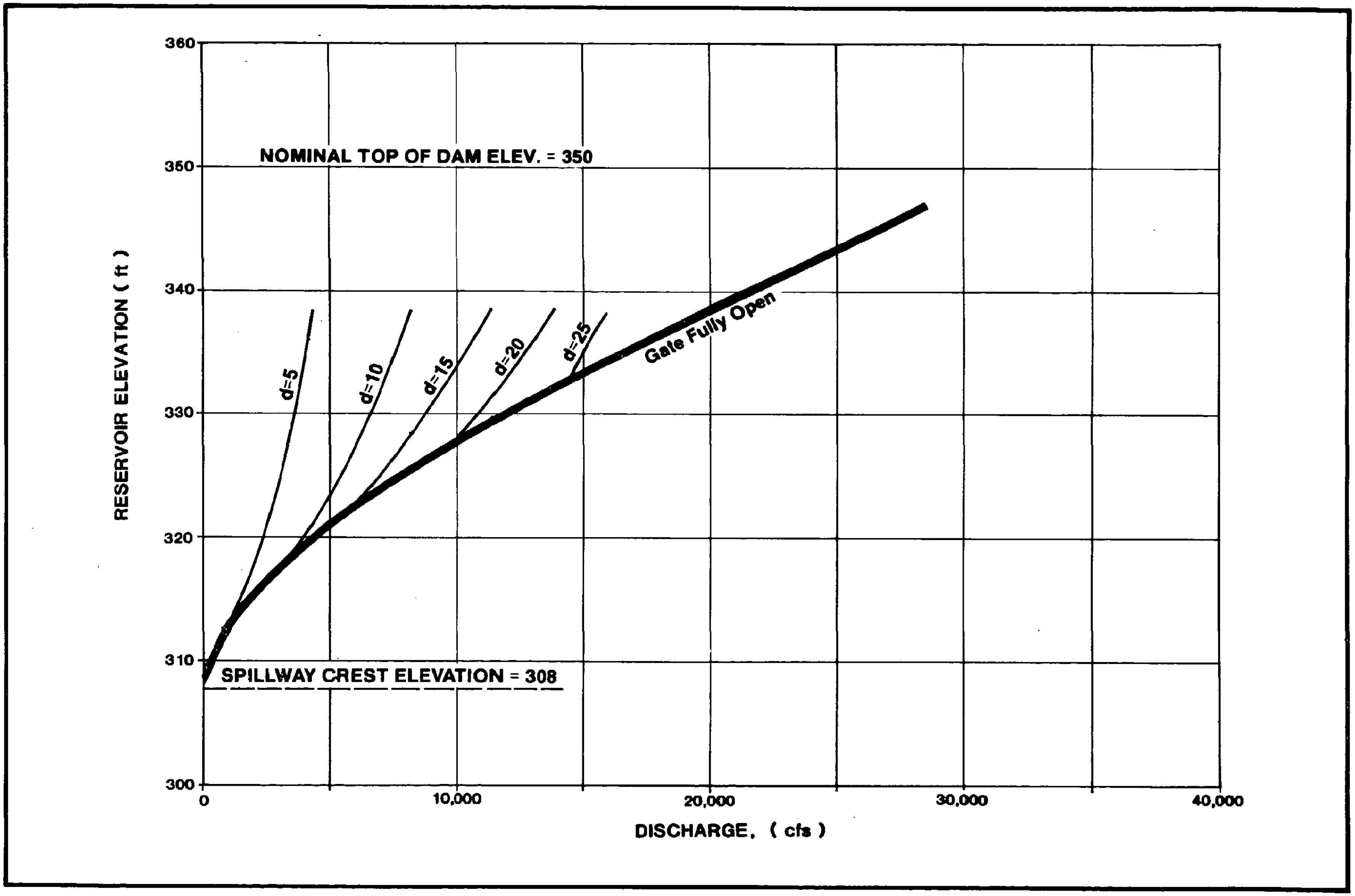


Figure 3-29
Lower Damsite
Area/Capacity Curves



NOTE: d = Gate Opening, ft.

Figure 3-30
Spillway Rating Curve
For a 30-Foot-Wide Gate

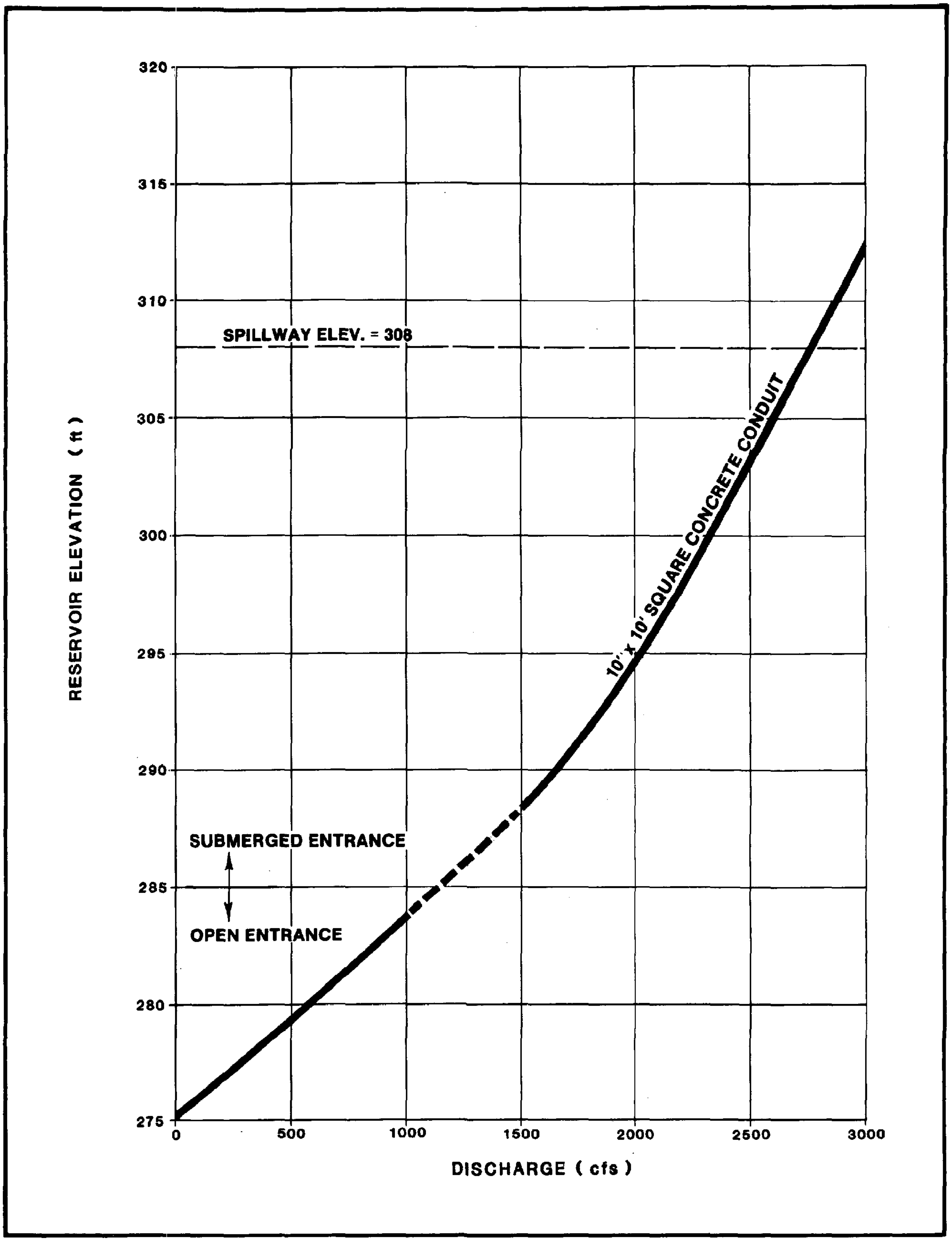


Figure 3-31
Low-Level Outlet Works Rating Curve for Conduit

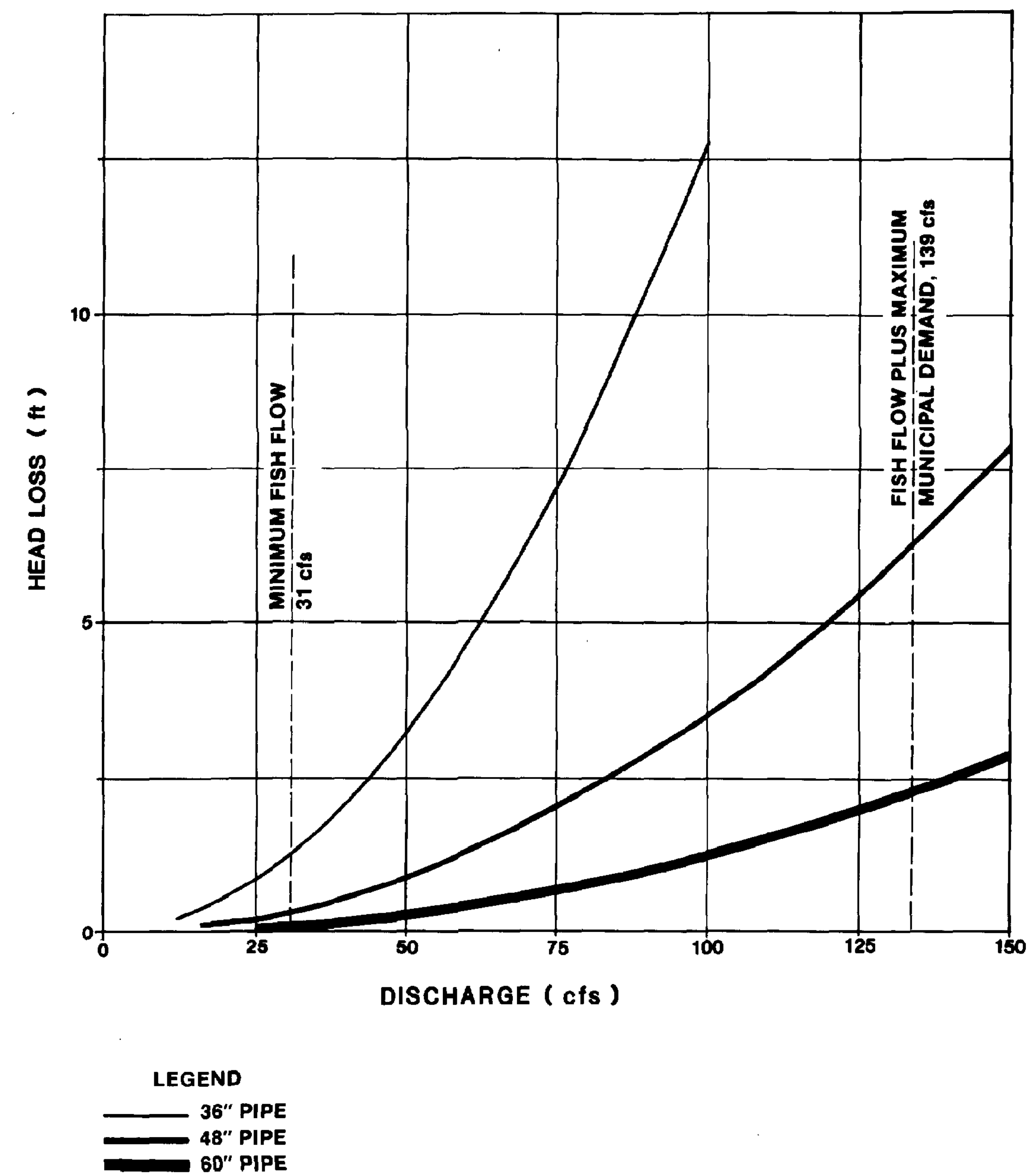


Figure 3-32
Low-Level Outlet Works Rating Curve for Pipes

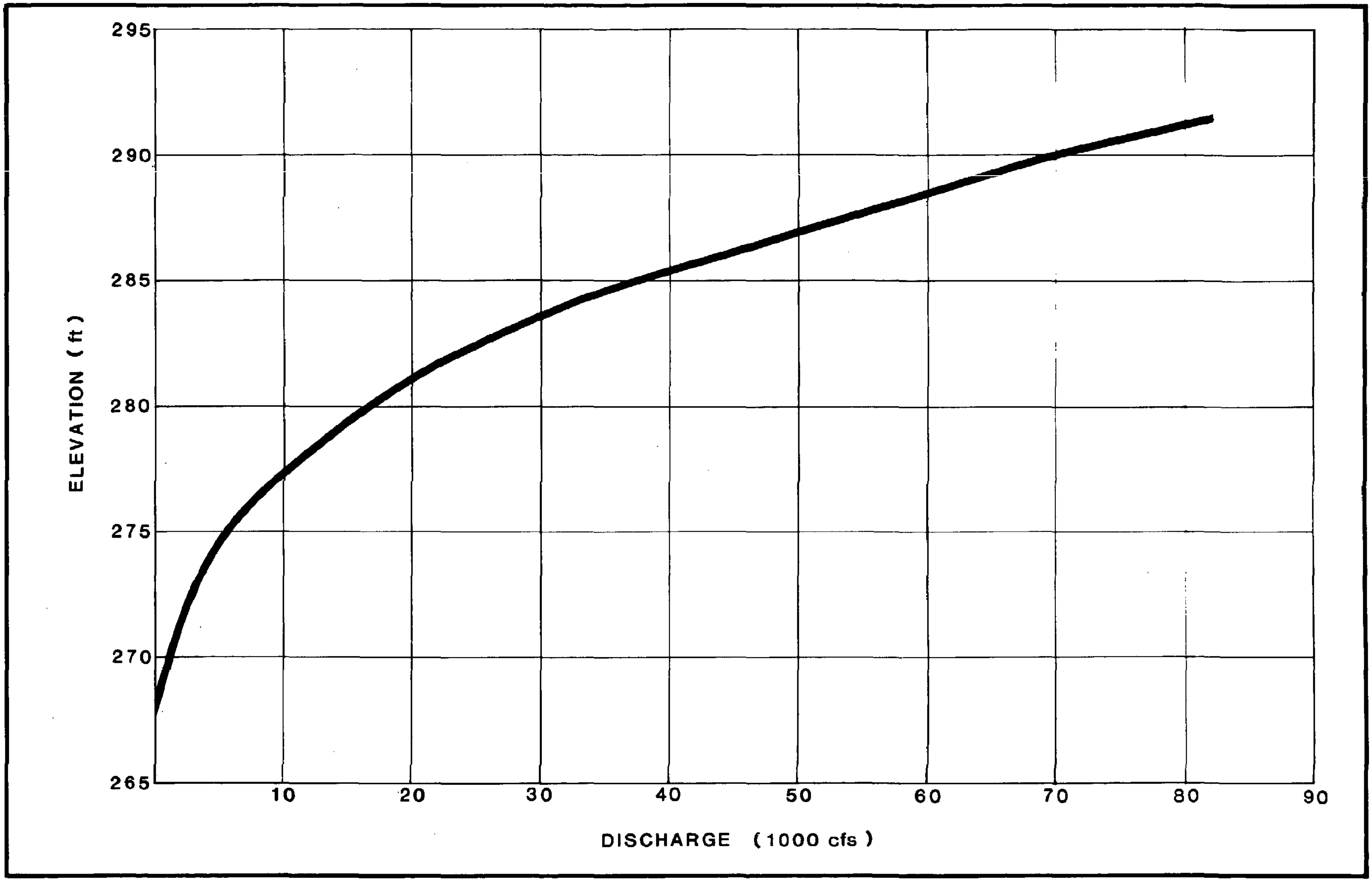


Figure 3-33
Lower Damsite
Tailwater Rating Curve

Chapter 4
GEOLOGY

REGIONAL GEOLOGY

The Eagle River damsite is located near the boundary of two major physiographic and geologic provinces: the Chugach Mountains surround it to the east, north, and south; and the Knik Arm of the Cook Inlet-Susitna Lowland is located just to the west. Strictly speaking, the site is within the Chugach Mountains. However, the base level established for the Eagle River by the nearby Knik Arm, as well as the glacial history of that area, have profoundly influenced the geologic history of the Eagle River Valley near the damsite.

Major geologic units in this area range in age from Permian to Quaternary (Clark, 1972; Schmoll and Dobrovolny, 1972; Zenone et al., 1974). These include older metamorphic and igneous rocks of Mesozoic and Paleozoic age (100 to 270 million years old), Tertiary sedimentary rocks (26 to 38 million years old), and Quaternary glacial and alluvial sediments (less than 2 million years old). The Quaternary glacial and alluvial sediments are discussed in the section Local Geology later in this chapter.

The Tertiary rocks belong to the Kenai group and are probably of Oligocene age (22.5 to 35 million years old). The tertiary rocks are separated from the older Mesozoic metamorphic rocks by the Knik Fault, a steeply-dipping-to-nearly-vertical fault that also divides the Cook Inlet Lowland from the Chugach Mountains. Paleozoic and Mesozoic rocks include the Jurassic (up to 190 million years old) to Cretaceous (up to 136 million years old) age Valdez and McHugh Groups, as well as older igneous and undifferentiated metamorphic rocks. The Valdez group is separated from the McHugh group by the Eagle River Thrust Fault, located several kilometers east of the Knik Fault.

These geologic units and their structural features are discussed in the following paragraphs.

Undifferentiated Metaplutonic, Metasedimentary, and Metavolcanic Rocks of Paleozoic or Mesozoic Age

These rocks are of uncertain age, possibly as old as Permian or as young as Jurassic. They outcrop mainly in the area near the mouth of the Eklutna River, but isolated outcrops are present as far south as the Eagle River. The outcrops include metamorphosed quartz diorite and gabbro, marble, siliceous argillite, metachert, and metasandstone. Typically, the rocks are of greenschist facies and are poorly fossiliferous.

McHugh Complex

Rocks of the McHugh complex are widespread in the project vicinity and probably underlie the damsite (beneath the glacial deposits). The McHugh complex consists of a metaclastic sequence and a metavolcanic sequence that are chaotically mixed together. The metaclastic sequence makes up the largest portion of the complex and consists of dark gray or green low grade metaclastic rocks, including siltstone, graywacke, arkose, and conglomeratic sandstone. The metavolcanic portion includes greenstone (basaltic composition), metachert, cherty argillite, and argillite. McHugh rocks are thought to be of late Jurassic to Cretaceous age (Clark, 1972).

Valdez Group

Rocks of the Valdez group are Jurassic to Cretaceous in age and are exposed over a large area in the Eagle River Valley upstream from the project (as well as elsewhere). Valdez rocks consist mostly of metagraywacke, metasiltstone, and argillite, with minor metaconglomerate. They are notably unfossiliferous and, generally, of lower greenschist facies metamorphic grade (Clark, 1972).

Igneous Rocks

These rocks include ultramafic, gabbroic, and felsic plutonic rocks that generally are late Paleozoic to Cretaceous in age. They mainly outcrop in the mountains southwest of the town of Eklutna, but isolated outcrops are also present in the project vicinity.

Sedimentary Rocks, Kenai Formation

These rocks are of probable Oligocene age and consist of nonmarine sandstone, siltstone, and claystone. They are exposed in a few areas along the Eagle River, downstream from the damsite.

Structure

Two major faults, the Knik Fault and the Eagle River Thrust Fault, divide the area into three tectonic blocks. Each of these blocks has a distinctive type of internal deformation.

Knik Fault. The Knik Fault is a steeply dipping to vertical sequence of faults, more properly called a fault zone. Its trace roughly parallels the Chugach Mountain front, where it is buried for much of its length by glacial debris. There is little topographic expression in most places along the fault, and no evidence of recent activity has been recorded. The fault is located about 2 miles west of the damsite.

Eagle River Thrust Fault. The Eagle River Thrust Fault is a complex zone of imbricate faulting that has moved McHugh complex rocks on top of younger Valdez group rocks along a low-angle fault. The Eagle River thrust is cut by local high-angle faults in many areas. No evidence of recent activity on the Eagle River thrust or associated high-angle faults has been recorded. The fault is located about 2 miles east of the damsite.

Deformation in Rocks West of the Knik Fault. Rocks in this block are characterized by well-developed schistosity and tight small-scale folding. Schistosity generally trends northeast with moderate to steep dips.

Deformation in the McHugh Complex Block. The McHugh complex rocks lie between the Knik and Eagle River Faults. These rocks have been affected by at least two generations of extensive, closely spaced shear fractures. The older set dips steeply and generally trends north-northeast, while the younger set has low dips with no distinct trend in folds.

Deformation in the Valdez Group. Valdez group rocks are exposed west of the Eagle River Thrust Fault. Deformation in the Valdez rocks is characterized by tight similar folds, slaty cleavage that runs approximately parallel to axial planes, and by numerous high- and low-angle faults. Folds trend generally north-northeast and have steeply dipping axial surfaces.

Tectonic Interpretation

According to the plate tectonics model, the convergence of the Pacific Oceanic plate with the North American Continental plate has resulted in the metamorphism, deformation, and accretion of portions of the oceanic plate onto the continent. The older metamorphic rocks located north and west of the Knik Fault are thought to be part of former oceanic coast that was accreted to the continent during Late Triassic or Early Jurassic time (180 to 150 million years ago). The ultramafic rocks, gabbros, metabasalts, and metacherts are characteristic of an ophiolite sequence, representing oceanic crust. The Jurassic plutonic rocks, which have intruded them, are often associated with such accretionary events.

The metaclastic rocks of the McHugh complex were probably originally deposited on oceanic crust, but derived from a rapidly eroding island arc system located inside the plate boundary on continental crust. The metavolcanic portions appear to have been oceanic crust. These materials were metamorphosed and accreted to the continent some time in the mid-Jurassic (140 million years ago).

Valdez group rocks appear to represent deep-water marine turbidites that were deposited on oceanic crust during the Cretaceous age. These rocks were deformed and accreted to the continent at the same time or just after the McHugh complex rocks were accreted.

A more detailed discussion of regional tectonics and faulting is presented in Exhibit C.

LOCAL GEOLOGY

The Eagle River Valley contains a complex assemblage of bedrock, glacial, alluvial, and colluvial materials. The bedrock units have been discussed previously. The glacial, alluvial, and colluvial deposits encountered during the exploration of the damsite and immediate vicinity all formed during Pleistocene time (within the last one million years). They have a complex interrelationship that has resulted from the interaction of depositional and erosional processes. The best way to understand the geologic history and existing geology of the damsite area is to look at the site's geologic features in terms of the processes that have formed them. Discussion of these processes is beyond the scope of this report; however, it may be helpful for those looking at the geologic maps or visiting the site to view the geology as having originated from the following processes:

Erosion: By glacial ice and stream waters

Transport: By glacial ice, stream waters, or under the influence of gravity (without water)

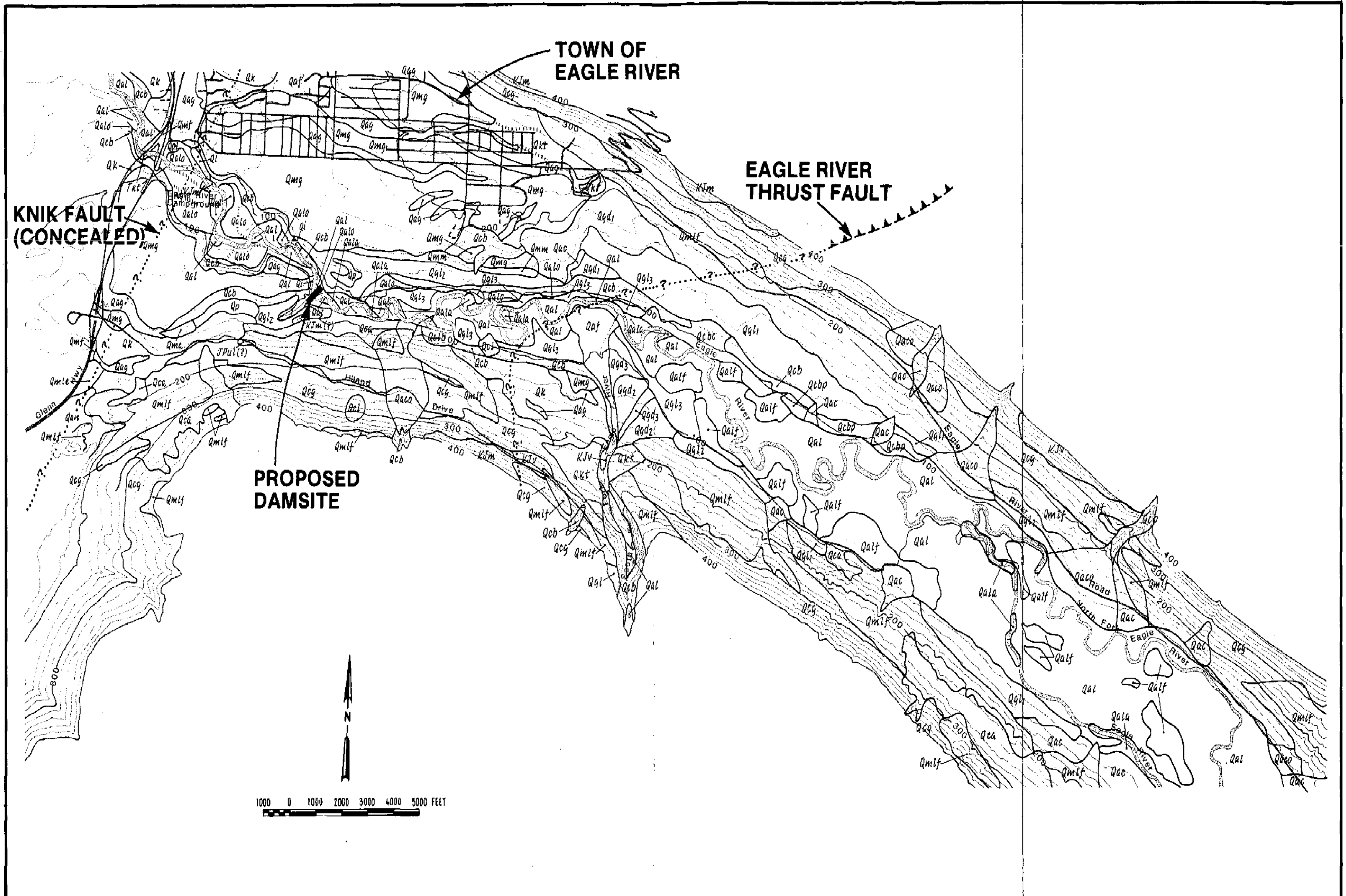
Deposition: From glacial ice, lakes and ponds, and streams and from downslope slumping or creep (variety of mass-wasting processes)

Weathering: Chiefly disintegration by ice-wedging and freeze-thaw processes, with minor oxidation and hydration

Geologic Mapping

The United States Geological Survey (Schmoll et al., 1980) has recently published a map showing geologic features in the Eagle River Valley from Glenn Highway to the upper end of the reservoir area. We have extracted pertinent sections of that map for inclusion in this report (Figure 4-1 and Table 4-1). The original scale of this map is 1:25000.

CH2M HILL geologists mapped the damsite area in more detail (1:1200) during the week of April 12, 1981. This map (Figure 4-2), should be viewed as preliminary and subject to revision and addition of detail during design.



NOTES: 1. Geology Taken from USGS Open File Report 80-890, Plate 1 by H.R. Schmoll, E. Dobrovolny, & C.A. Gardner.
2. See Table 4-1 for Description of Geologic Units.

Figure 4-1
Regional Geologic Map

Table 4-1
LEGEND FOR MAP UNITS IN FIGURE 4-1

Alluvial Deposits

Qala:	Alluvium in active flood plain: sand, gravel, and cobbles
Qal:	Alluvium in lowest terraces: sand, gravel, and cobbles
Qalo:	Older alluvium on higher terraces: sand, gravel, and cobbles
Qag:	Alluvium deposited in channels and fans during the waning phases of glaciation: chiefly sand and gravel
Qaf:	Alluvial fan deposits formed where streams enter the main valley: sand and gravel
Qac:	Alluvial cone deposits formed where small tributaries enter the valley and abruptly decrease in gradient: sand and gravel
Qaco:	Older alluvial: cone deposits, graded to a higher base level than at present--sand and gravel

Glacial Deposits

Qmlf:	Lateral and terminal moraine inferred to be equivalent in age to the moraine near Fort Richardson: chiefly diamicton (till)
Qmg:	Ground moraine: chiefly composed of diamicton (till)
Qmm:	Ground and lateral moraine that appears to have been modified by contact with glacial lakes

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Note: The information in this index was derived entirely from USGS Open File Report 80-890 (1980) by Henry S. Schmoll, Ernest Dobrovolny, and Cynthia A. Gardner. Omissions in this index are the responsibility of CH2M HILL. For more complete descriptions, see Open File Report 80-890.

Table 4-1
(Continued)

Glacio-Alluvial, Lacustrine, and Deltaic Deposits

Qkt:	Kame terrace deposits formed in tributary valleys blocked by glacier ice in the main valley: sand and gravel
Qk:	Kame and related ice-contact deposits laid down in or around glacial ice: sand and gravel
Qgl_3: Qgl_2: Qgl_1:	Glaciolacustrine deposits: chiefly silt, clay, and very fine sand: subscripts indicate different episodes from 1 (oldest) to 3 (youngest)
Qgd_3: Qgd_2: Qgd_1:	Delta deposits formed marginal to former lakes in Eagle River Valley: chiefly sand and gravel: subscripts indicate different episodes from 1 (oldest) to 3 (youngest)

Pond Deposits

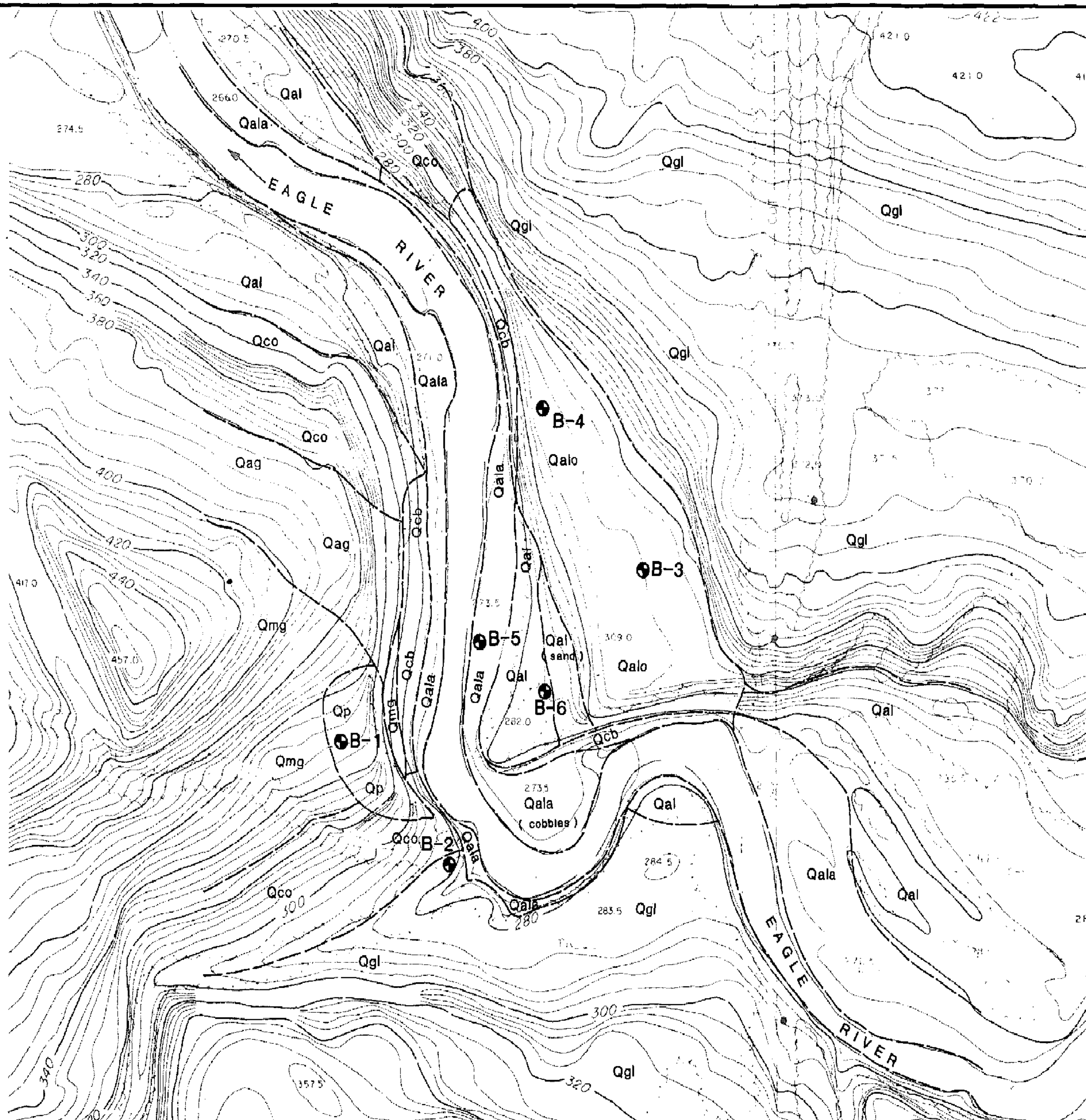
Qp:	Postglacial pond deposits: clay, silt, and peat
Qi:	Interglacial pond deposits: chiefly silt and clay: generally only a few meters in thickness

Colluvial Deposits

Qca:	Colluvial: alluvial mixed material derived from weathering of bedrock upslope and moved primarily by gravity, with minor transport by water
Qcg:	Mixed colluvium and glacial deposits: similar to Qca, but includes morainal deposits
Qcb:	Colluvium developed in surficial deposits on river bluffs and canyon walls: chiefly diamicton with interlayers of finer material.
Qcbp:	Areas of poorly defined bluffs where fine grained material has slumped to obscure the morphology of the bluffs

Table 4-1
(Continued)

	Landslide Deposits, Generally Consisting of Diamicton
Qcl:	Rapidly emplaced by debris avalanching
Qcle:	Emplaced slowly by earth flow
Qclb:	Large masses of bedrock that have moved downslope intact
Qcll:	Landslide debris-- possibly modified by lacustrine erosion and deposition
Qcld:	Landslide debris on subdued terrain, possibly modified by lacustrine and deltaic processes
Qcs:	Solifluction deposits formed by downslope creep
Anthropogenic Deposits	
Qmf:	Engineering highway fills
Qma:	Area altered by man--southwest of proposed damsite includes sanitary landfill
Bedrock Units	
Tkt:	Tyonek formation (Miocene and Oligocene): chiefly nonmarine sandstone, siltstone, and coal
KJv:	Valdez group (Cretaceous): chiefly argillite, siltite, and meta graywacke
KJm:	McHugh complex (Cretaceous and/or Upper Jurassic): chiefly massive, weakly metamorphosed sandstone and conglomeratic sandstone with minor metavolcanic rocks
JPu:	Igneous rocks (Jurassic to Permean): chiefly gabbro



LEGEND

GEOLOGIC MAP UNITS

Qmg	Ground moraine
Qag	Outwash gravels
Qgl	Glaciolacustrine deposits
Qalo	Older alluvial deposits
Qco	Older colluvial deposits
Qp	Pond deposits
Qal	Younger alluvial deposits
Qcb	Active colluvial deposits
Qala	Recent alluvium in stream flood plains

For more detailed descriptions, see the text.

⊕ Boring Location

N

SCALE: IN FEET

100 50 0 100

Figure 4-2
Damsite Geologic Map

Geologic Units

This section describes the geologic units that were encountered during our exploration of the damsite and surrounding vicinity (approximately a one-half-mile radius). They are described in their general chronological order, starting with the oldest first. Chronologically and spatially, many units overlap and grade from one to another; therefore, their sequence is a generalization. The symbols given with each unit's description (e.g., Qmlf) are those used in Figure 4-2.

Lateral Moraine (Qmlf). These deposits are chiefly till or diamicton (mixtures of all size particles) formed as lateral moraine and located on the south valley side above the proposed damsite. Schmoll et al. (1980) infer that they are equivalent in age to the lateral moraines in the Fort Richardson area.

Ground Moraine (Qmg). Equivalent in age to the lateral moraine deposits, this material was probably laid down beneath the glacier or is an ablation deposit formed during the glacial recession. In the project vicinity these deposits were observed to be a coarse diamicton (till) that is well-graded from fine sand to 12-inch boulders, contains little silt or clay (tested at 2.5 percent), and is very dense. These ground moraine deposits show a very faint degree of stratification. The left dam abutment is partially composed of this unit. The ridge west of the damsite and the upper valley north of the damsite contain large deposits of ground moraine.

Outwash Sand and Gravel (Qag). These deposits overlie the ground moraine material located in the left abutment (downstream side) and thus are thought to be younger. This relationship is clearly shown in the high river cutbank at the left abutment. The deposits consist of well-graded fine or medium sand up to coarse gravel. They contain only about 5 percent material that is larger than 4 inches in diameter and about 2 percent silt and clay fraction. They show sufficient stratification to indicate that they were deposited by flowing water and were probably formed in channels or alluvial fans during the waning phases of glaciation (Schmoll et al., 1980). The hillside downstream on the left side contains a considerable amount of this material.

Glaciolacustrine Deposits (Qgl). These deposits consist of silt, clayey silt, and sandy silt, or clay, as noted in the laboratory test results of materials from borings B-5 and B-6 (Chapter 5) and from borrow sample 11SU (Exhibits D and E). They contain slight to pronounced stratification, with layers ranging from 1/4 inch to several inches in thickness. Some layers are nearly all fine sand, while others have no sand. They formed in glacial lakes that developed as a consequence of the valley outlet channels being dammed downstream, probably by glacial ice. These glacial lake deposits are extensive in the damsite vicinity, both northeast and south of the site, as well as partially beneath it. Information from our borings indicates this material is present beneath the entire right side, center, and at least on the upstream half of the left side of the dam. Throughout the valley, three different episodes of lacustrine deposition have been mapped; however, the deposits near the damsite appear to be all of the second episode. Some third-episode deposits may be present beneath alluvial deposits on the low river terraces in the reservoir.

Older Alluvial Deposits (Qalo). Older alluvial deposits are younger than the lake deposits because they are present in eroded areas of former lake deposits and overlie these lake deposits. They generally consist of slightly stratified well-graded sand and gravel, and probably represent channel fill. They are present in the right abutment and spillway areas, as well as several locations upstream and downstream. Generally, they are higher and thicker than the younger alluvial deposits.

Older Colluvial Deposits (Qco). These materials are derived from nearby glacial or alluvial deposits and form wedge-shaped deposits downhill, moving chiefly under the force of gravity and without running water. They are present along many slopes in the damsite vicinity, notably in the left abutment (upstream portion) and along the north side of the valley that joins the main Eagle River Valley just above the damsite, essentially mantling the left abutment upstream. They have been observed to be variable in composition from rather dirty diamicton to poorly graded sand, gravel, or clay, depending largely on their source. Because these deposits are older, they are well-vegetated and show very slow movement at the present time.

Pond Deposits (Qp). Pond deposits are present at several locations near the damsite, including overlying the left abutment's glacial deposits. They are similar in composition and stratification to glacial lake materials, but younger and confined to local environments. They are generally thin (3 to 10 feet thick) and usually show contorted bedding, indicating that they are ice-contact deposits.

Younger Alluvial Deposits (Qal). These deposits are similar in nature to the older alluvial deposits, but are younger and generally within the river's present floodplain. They are stabilized by moderate to extensive vegetation and are located in gravel point bars and channel fills.

Active Colluvial Deposits (Qcb). These are deposits that are presently forming under the influence of gravity along the base of moderate to steep slopes. Most notable of these deposits in the damsite vicinity is the large colluvial wedge at the base of the river cutbank along the entire left abutment. Freeze-thaw cycles cause loosening of ground moraine, outwash gravel, and older colluvial materials in this cutbank. These cycles allow gravity to move the material downslope, forming the characteristic wedge shape.

Recent Alluvium (Qala). These are materials actively transported by the river during periods of high water. They consist of sand, gravel, and cobbles that form temporary bar and bank deposits. Some sand also moves as streambed traction load, even under normal flows. Recent alluvial deposits generally were observed to have a low percentage of silt and clay fraction because silt and clay tend to stay in suspension. They are for the most part unvegetated.

Generalized Summary of Local Geologic Events

Evidence indicates there were one or more very extensive early Pleistocene glacial episodes in the Eagle River Valley. Judging by the location of lateral moraines on the valley sides, these early glaciers must have been up to 2 miles wide and 2,000 to 2,500 feet thick in the project vicinity. Some of the ground moraine deposits probably originated during these early episodes. Most of the lower moraine deposits probably were the result of later Wisconsin Stage glaciation, which was somewhat less extensive than the earliest episodes. Deposits of lateral and ground moraine, and other units, probably filled across most of the valley, at least up to about elevation 650 feet in the area downstream from the damsite. The outlet for the Eagle River was probably along the south side of the valley, through the present tributary valley located upstream and immediately south and west of the damsite. This allowed the river to flow out somewhere near the Hiland Road-Glenn Highway junction. Periodically, this outlet or other outlets must have been blocked by glacial ice downstream, thus creating the three glacial lakes of which we find evidence today at various levels in the valley.

Between these episodes of lacustrine deposition there were continued erosion and perhaps even minor glacial advances and retreats.

Erosion and deposition by stream waters alternated and created the different levels of terrace deposits we see today, as well as the meander scars visible in the valley walls. Drainage was established through the existing outlet downstream from the damsite, probably sometime after the second glacial lake formed. Colluvial processes were active from the earliest glacial episodes, and evidence of very old to recent deposits is present. , At the present time the river appears to be actively downcutting, and erosion is the dominant process in the project vicinity.

GEOLOGIC HAZARDS

We conducted a preliminary evaluation of geologic hazards to the damsite and reservoir area. This evaluation was based on air photo interpretation, ground reconnaissance, drilling information, and the preliminary seismic evaluation made by our subconsultant Lindvall, Richter & Associates (Exhibit C). The geologic hazards we have addressed include earthquake damage to the dam, fault rupture, landslides, liquefaction, and soil settlement.

Earthquake Damage to the Dam

Damage to the dam embankment caused by seismic shaking is a possibility. According to the preliminary seismic evaluation (Exhibit C), the dam should be designed for a peak acceleration of 0.4g and an effective peak acceleration of 0.33g. Measures to mitigate potential earthquake damage to the dam and appurtenant structures are discussed in Chapter 8.

Fault Rupture

Our exploration did not reveal the presence of any faults, either active or inactive, underlying the damsite. The Eagle River Thrust Fault, considered inactive, crosses the proposed reservoir site about 1 to 2 miles east of the damsite. We know of no evidence to indicate that this fault or any other fault presents a surface rupture hazard to the damsite.

Landslides

There are two types of potential landslide hazards to the project: earthquake-induced landslides and landslides generated by rapid drawdown of the reservoir. The reservoir area contains very heterogeneous deposits, ranging from fine-grained glacial lake sediments to coarse glacial till and alluvial and colluvial deposits. Some older landslide deposits are also present in the reservoir area. Under saturated conditions, an earthquake might induce a landslide in some of the less consolidated colluvial desposits or cause older landslides to move. Lacustrine deposits (fine-grained) might become unstable under rapid drawdown conditions; the effects of a lacustrine deposit landslide on the damsite would depend on the slide's location, volume, and rate of entry into the reservoir. A study of the stability of various deposits in the reservoir area is beyond the scope of work for this project. A stability evaluation should be made during final design to assure that the dam and reservoir would have an adequate margin of safety to handle potential landslide problems.

Earthquake-Induced Liquefaction or Settlement

Liquefaction and failure of soils as a result of seismic shaking generally requires three elements: (1) sufficiently high, repeated ground accelerations, (2) soil saturation, and (3) fine sandy or silty soil that is not highly densified. The level of seismic shaking may be high enough at some time during the Eagle River dam's lifetime to cause liquefaction of relatively loose saturated soils. However, the materials to be used for dam construction would be highly compacted, which would make liquefaction or settlement of the embankment highly improbable. The very dense condition of the dam's foundation materials would also preclude liquefaction. Settlement from earthquake shaking should also be very unlikely because of the proposed density of the materials. (Further discussion is presented in Chapter 8.)

PREVIOUS GEOLOGIC STUDIES

Our review of published literature indicates that a number of previous geologic studies of the Eagle River Valley have been made.

The earliest study we found was done by A. F. Bateman in 1948, though not published by the U.S. Geological Survey (USGS) until 1980 (Bateman, 1980). This study is an evaluation of the Eagle River Valley for potential damsite locations. Two potential sites were selected, and a reconnaissance evaluation of the site geologic conditions was made. One of Bateman's sites is nearly the same as the present site, although slightly downstream.

In 1974 the USGS published a land use planning report on geology and groundwater in the Eagle River-Chugiak Area (Zenone et al., 1974). This report discusses the area's geology and its impact on urban and groundwater development.

An electrical resistivity and seismic refraction survey was conducted by the USGS in 1979 to determine the depth to bedrock in the middle valley area of the Eagle River Valley (above the proposed reservoir). Their study indicated that 350 to 450 feet of unconsolidated sediments overlie bedrock. They made an additional electrical resistivity sounding in the tributary valley located south of the proposed damsite. Their findings suggest the bedrock surface is near elevation 50 feet. This tends to confirm that this valley may have once been the major outlet for the Eagle River.

Probably most valuable of the available publications is the preliminary geologic map of the middle portion of the valley, published by the USGS in 1980 (Schmoll et al., 1980). This map (see Figure 4-1) provides a detailed breakdown of the valley's complex geology and is accompanied by a thorough description of mapped units.

Chapter 5
FIELD EXPLORATION

FIELD EXPLORATION ACTIVITIES

Geotechnical field exploration activities consisted of a geologic reconnaissance of the site, drilling, and surface bulk sampling of materials in river cutbanks.

Geologic Reconnaissance

An initial geologic reconnaissance of the project site was made at the time of drilling operations (January 1981). Later, during the week of April 12, 1981, a more thorough reconnaissance of the damsite and surrounding area was made, and a geologic map of the damsite area was prepared. The purpose of this work was to locate potential borrow sources and to look for geologic hazards that could affect the conceptual design of the project.

In addition, color and black-and-white aerial photographs taken in September 1979 (Air Photo Tech, Inc., 1979) were used for reconnaissance of a much larger area in the Eagle River Valley. These photographs helped to evaluate the potential geologic hazard to the project and to identify the sequence of geologic events described in the previous chapter.

Drilling

Between January 13 and February 11, 1981, six borings were drilled in the vicinity of the proposed dam to acquire subsurface information at the proposed locations of the spillway, inlet tower, and main embankment. The drilling was done by Exploration Supply and Equipment, Inc., of Anchorage. They used an Acker MP-4 drill rig mounted on a Nodwell chassis; mud rotary methods were employed. Biodegradable drilling mud "Revert" was used at locations near the river, and bentonite mud was used at other locations. The locations of the borings are shown in Figure 5-1. The boring logs are shown on Figures 5-2 and 5-3.

Drilling and sampling operations were directed and monitored by a CH2M HILL engineering geologist. Samples were visually classified in approximate accordance with American Society for Testing and Materials, Standard D 2488, "Visual-Manual Procedure for Description of Soils". Selected samples were taken to the laboratory of Harding-Lawson Associates for classification tests and engineering properties tests. The remaining samples were stored.

A piezometer was installed in boring B-6 to monitor ground water in an aquifer encountered at the 54- to 60-foot depth. The piezometer was made of 3/4-inch-diameter PVC pipe. The pipe was capped on the bottom and slotted with a hacksaw at 3-inch intervals for the bottom 20 feet. It was surrounded with pea gravel for the bottom 25 feet and sealed with clay for the top 35 feet. A 1/2-inch-diameter steel pipe was installed at the surface to help reduce vandalism damage. The piezometer flowed 1 to 2 gpm at the surface. Water would rise to approximately 2 feet above the ground surface when a riser was connected to the top of the piezometer. When the piezometer was checked again in April 1981, flow was continuing at the same rate.

Bulk Sampling of Borrow Materials

Much of the potential material for the dam construction is sand, gravel, and cobbles. Samples of this material cannot be properly obtained for testing by drilling. Bulk samples (60 to 100 pounds) must be taken either from river cutbanks or from backhoe test pits. Backhoe excavations were not necessary because of the easy accessibility of most potential borrow materials at the river cutbanks.

Initial sampling of representative materials in the river cutbanks was done on February 11, 1981, followed by additional sampling during the week of April 12, 1981. Materials sampled include ground moraine deposits, outwash sand and gravel, older alluvial deposits, recent alluvium, and glaciolacustrine deposits. The laboratory test results in Exhibits D and E provide descriptions of the properties of these materials.

SUBSURFACE CONDITIONS

The descriptions of subsurface conditions that follow are based on information obtained from our January and February 1981 drilling program, observation of river cutbanks, and from USGS studies of the Eagle River area. Additional field work during design would be required to confirm these findings. Because of the great variability of material types, both laterally and vertically, identified in the area, the descriptions should be considered generalizations, and may not be accurate at all points.

Right Abutment (North Side) and Spillway Area

The surface in this area is covered by a 1- to 3-foot-thick mat of fibrous peat or muskeg. Coarse sandy cobbly gravel (Unified Soil Classification of GW) is encountered beneath this mat down to approximately elevation 255 or 260. This material contains 2 to 10 percent silty matrix. The geomorphology and material type of this area suggest that, at least near the surface, this material is an older alluvial deposit. Possibly the alluvium changes to a coarse glacial outwash deposit at some intermediate depth, but this has not been confirmed.

The sandy cobbly gravel is underlain by approximately 10 feet of clean medium sand (SP), with a top elevation of 255 to 260 feet.

This sand is interpreted to be an older stream deposit. Below elevation 245 or 250 feet, sandy silt was encountered. This material graded into silt (ML) within 5 to 10 more feet of depth. This silt is bluish-gray, very stiff to hard, and has low plasticity. Some parts of this layer are predominantly clay (CL). The layer extended past the greatest depth drilled (elevation 222 feet).

Occasional thin (1/2-inch to 1-inch) seams of fine sand were found, as well as occasional single cobbles. This material almost certainly is a glacial lake deposit, with the cobbles possibly having been ice-rafted.

A similar sequence of deposits is located in a cutbank several hundred feet downstream from the proposed right abutment and spillway location. In that exposure, the slumped and contorted nature of the sand and silt layers suggests that the deposits are ice-contact sediments.

Main Embankment (Middle) and Control Tower Area

The surface in this area is covered by up to 3 feet of fibrous peat (muskeg) outside the zone of active river erosion; within this zone fresh river alluvial deposits are exposed. These river alluvial deposits also underlie the muskeg and extend down to approximately elevation 260 or 265 feet. This material consists of interlayered mixed silt, sand, gravel, and cobbles. It varies from well-graded to poorly graded (GW/GP) and is unconsolidated.

Below the moderately coarse alluvial deposit, 2 to 5 feet of fine alluvial sand (SP) was encountered. This type of material is also present in a terrace that is about six feet higher than the terrace on which boring B-6 is located. Below the sand, at about elevation 260 or 265 feet, the same layer of silt described in the previous section was encountered.

This material extends down to approximately elevation 230. Within that layer, between about elevations 245 and 248 feet, is a 3-foot-thick layer of gravel and cobbles. At about elevation 230 feet, there is a gravel and cobble layer, with a clay matrix, about 8 feet thick. This layer contains minor artesian pressure. A piezometer was installed in boring B-6 to monitor the artesian pressure in this layer. This boring reached material that is probably bedrock at elevation 222 feet, and ended at about elevation 218 feet.

Left Abutment (South Side)

The left abutment contains a variety of glacial deposits. As on the right side, the surface is covered by 1 to 3 feet of muskeg. Underlying this is a 5- to 20-foot-thick slumped wedge of sandy cobbly silt (thicker toward the upstream side), probably an ice-contact glacial pond deposit. Underlying this deposit is a till deposit of variable thickness that appears to extend nearly the full height of the proposed embankment at its centerline. This material is a dense, poorly stratified, well-graded sandy cobbly gravel (GW) with only about 2.5 percent material passing the No. 200 standard sieve. This deposit is interpreted to be a glacial ground moraine that has been dissected by the Eagle River. Overlying the till on the downstream side and in partial contact with the left dam abutment is an outwash deposit of stratified silty sandy gravel. This material consists of clean sand and gravel, with only a little material larger than 4 inches in diameter. A wedge of older colluvium is present in the upstream portion of the left abutment and is derived from and transitional to the ground moraine located in the abutment center.

Information from boring B-2 indicates that silty sand and gravel extend down to about elevation 260 feet and, below this, sand and gravel, coarse gravel, cobbles, and sand with a silt matrix extend to at least elevation 232 feet. The latter layer appears to correlate with the silt layer identified in the right abutment and mid-embankment areas; however, beneath the left abutment this layer contains considerable amounts of coarse gravel, cobbles, and sand. This fact suggests that the left abutment may overlie the edge of a former glacial lake, where considerable colluvium was deposited along with the lacustrine silts. What is probably bedrock was reached at elevation 232 feet.

Bedrock

Information from borings B-2 and B-6 suggests that bedrock underlies the damsite at about elevation 220 to 230 feet. A water well was drilled 700 feet southeast of the damsite as a part of Task 1 of this study. This well encountered bedrock at elevation 244 feet. These three borings suggest that the bedrock surface dips slightly east of north at about 2 degrees. Rock cuttings from each of the borings were composed of similar material. The cuttings appeared to be metagraywacke, probably from the McHugh complex. A previous boring drilled by Retherford Associates (1966) north of the dam penetrated 240 feet and did not encounter rock.

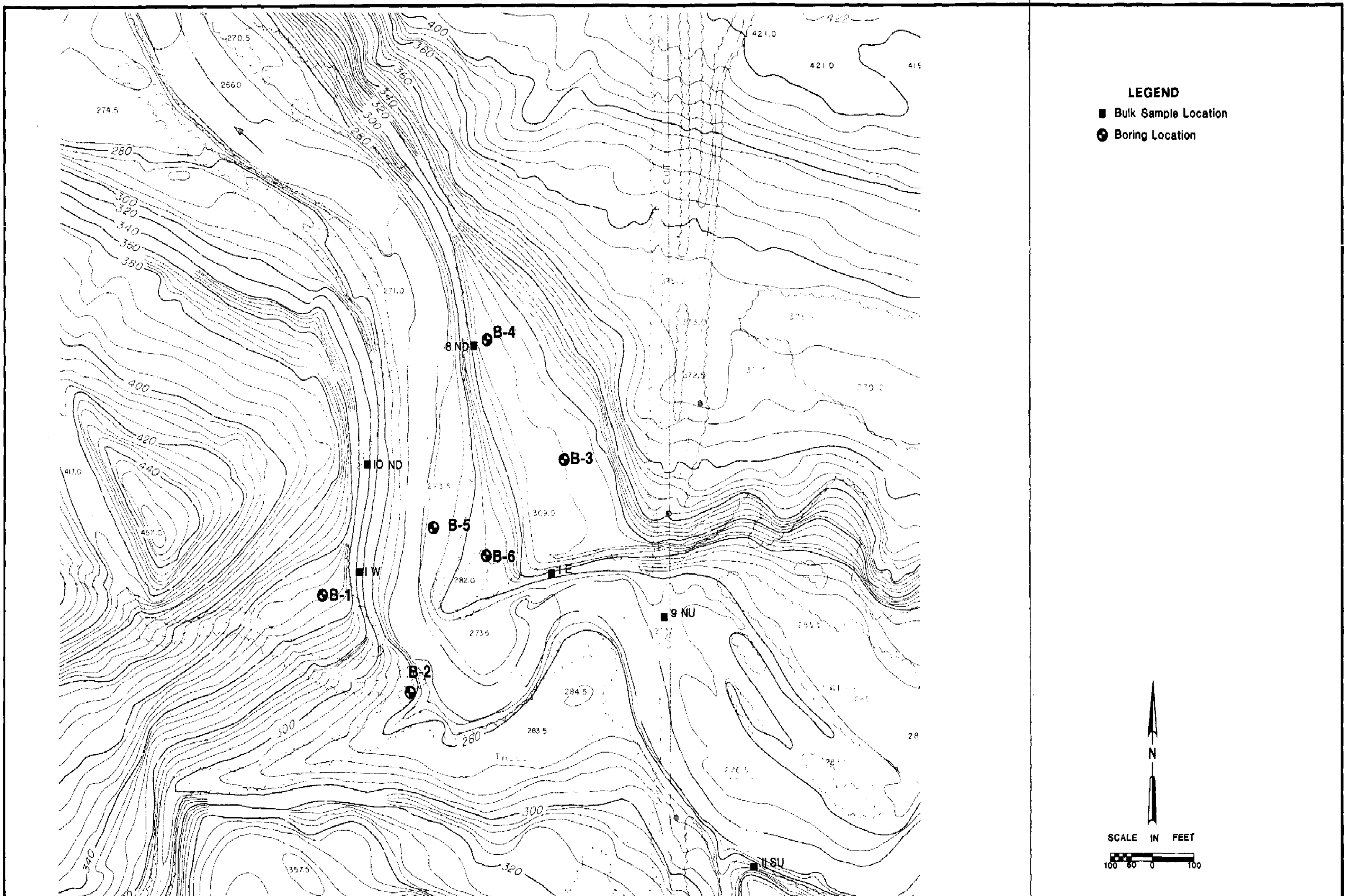


Figure 5-1
Exploration Plan

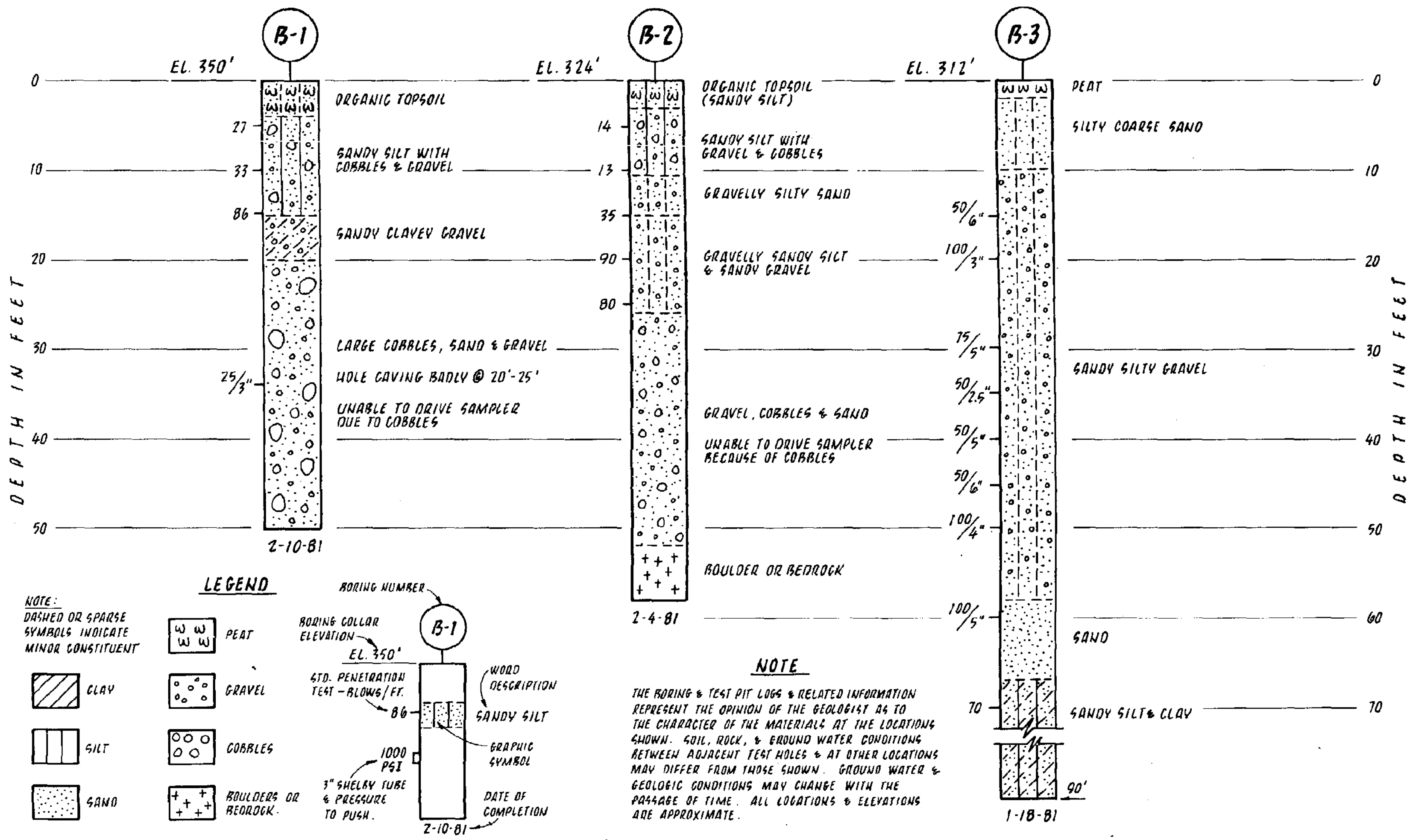


Figure 5-2
Damsite Boring Logs
B-1 Through B-3

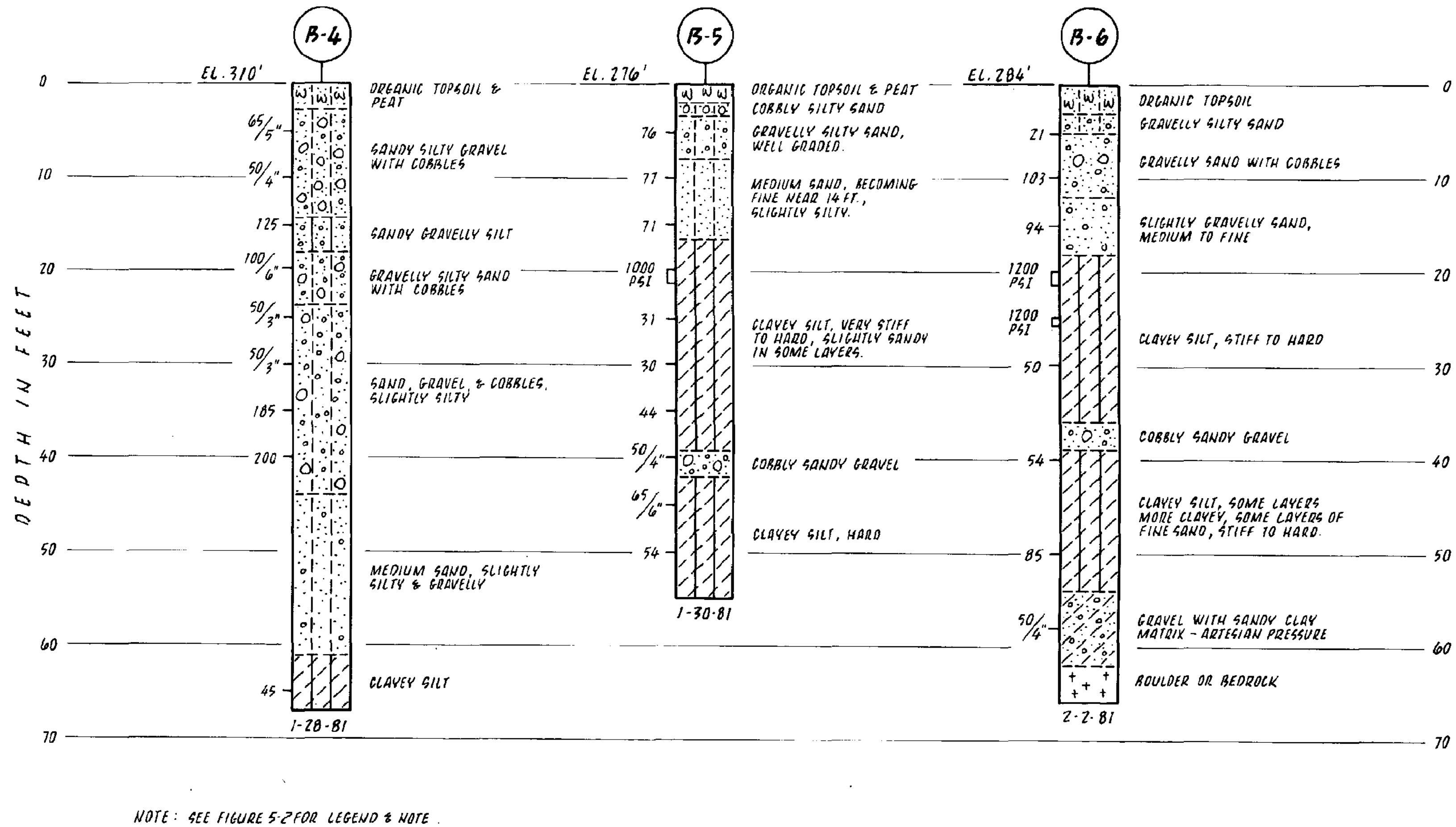


Figure 5-3
Damsite Boring Logs
B-4 Through B-6

Chapter 6
LABORATORY TESTING

This chapter briefly describes the laboratory tests that were performed on soil samples obtained during the field exploration. The testing is divided into two categories, classification testing and engineering properties testing. Classification tests were performed to broadly categorize the soils on the basis of their engineering properties. These tests included Atterberg limits, natural moisture content, grain size analysis, and specific gravity. Engineering properties were determined by triaxial shear test and by unconfined compression, consolidation, and compaction testing.

The tests were conducted by both CH2M HILL and Harding-Lawson Associates. The results of the testing performed by Harding-Lawson Associates are presented in Exhibit D, and the CH2M HILL test results are presented in Exhibit E. A summary of all laboratory test results is presented in Table 6-1.

CLASSIFICATION TESTS

Atterberg Limits

Atterberg limits were determined according to ASTM D423 and D424 on selected samples. The results are shown in on Plate 3 of Exhibit D and Figure E-1 of Exhibit E. In general, the results of Atterberg limits tests conducted on different samples of a given type of plastic soil will plot in a group generally parallel to the "A-line" shown in the figures.

The samples tested for this study fall into two general soil groups. First, a low-plasticity silt that plots just below and parallel to the A-line; these samples represent the silt found at about 18 feet below the valley bottom. The second soil group is a low-plasticity clay that plots just above and parallel to the A-line; these samples represent a lower, slightly more plastic facies of the low-plasticity glacial deposits beneath the valley bottom. These two groups, while technically classified as different soil types, have been combined for the purpose of analysis because of their proximity to the A-line and their similar characteristics.

Natural Moisture Content

Natural moisture content determinations of a number of soil samples were made according to ASTM D2216. The moisture content of the soil would fluctuate with the seasons, so these results may not be representative of the moisture content during construction.

Table 6-1
EAGLE RIVER DAM PROJECT SUMMARY OF LABORATORY SOIL TESTS

Sample No.[a]	Depth (ft)	Visual Description	Unified Soil Classification	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Percent Passing No. 200 Sieve	Specific Gravity	Shear Strength (psf)	Effective Internal Angle of Friction (deg)	Comp. Index
B-1 SS-1&2[b]	5 & 10	Sandy Silt	ML	7	-	-	-	92.5	-	-	-	-
B-2 SS-1	5	Sandy Silt	ML	26	-	-	-	82.5	-	-	-	-
B-3 SS-12	85	Sandy Silt	ML	30	39	26	13	-	-	-	-	-
B-4 SS-7,8,&9[b]	39,40,&45	Silty Gravelly Sand	SP-SM	12	-	-	-	9.0	-	-	-	-
B-5 SS-1&2[b]	5 & 10	Silty Gravelly Sand	SP-SM	15	-	-	-	8.0	-	-	-	-
B-5 ST-1	20	Silt	ML	24	33	26	7	99.8	2.75	-	38	.05
B-5 SS-4	25	Silt	ML	27	41	27	14	-	-	1230	-	-
B-5 SS-5	30	Silt	ML	32	-	-	-	-	-	2520	-	-
B-5 SS-6	35	Silt	ML	28	-	-	-	96.5	-	-	-	-
B-5 SS-8	45	Clay	CL	22	36	21	15	-	-	-	-	-
B-6 SS-1,2,&3[b]	5,10,&15	Silty Gravelly Sand	SP-SM	14	-	-	-	9.5	-	-	-	-
B-6 ST-1	20	Silt	ML	22	-	-	NP[c]	99.9	2.72	-	42	.04
B-6 ST-2	25	Silt	ML	26	39	27	12	99.6	2.74	-	38	.07
B-6 SS-4	30	Silt	ML	29	49	28	21	-	-	4300	-	-
B-6 SS-5	40	Clay	CL	25	36	22	14	-	-	4530	-	-
1W	0	Sandy Gravel	GW	-	-	-	-	2.5	-	-	-	-
1E	0	Sandy Gravel	GW	-	-	-	-	2.9	-	-	-	-

[a]Boring and sample numbers, except 1E and 1W which are bulk samples (see Figure 5-1 for location).
[b]Combined.
[c]Nonplastic.
Note: SS = split spoon sample.
ST = seamless steel tube sample.

Grain Size Analyses

Grain size analyses were performed on selected soil samples to aid in soil classification and to provide information that can be used to estimate soil permeability. These analyses were done by mechanical (ASTM C136) and hydrometer (ASTM D422) methods. Additional samples were also tested in accordance with ASTM C117 to determine the percent passing the No. 200 sieve. A summary of the grain-size analyses is shown in Plates 2 and 18 of Exhibit D.

Specific Gravity

The specific gravity of solids for selected samples was determined according to ASTM D854. These values were used in computations for engineering properties tests.

ENGINEERING PROPERTIES TESTS

Triaxial Shear Test

Consolidated-undrained triaxial shear tests with pore pressure measurements were performed according to ASTM D2850 to provide an estimate of the effective stress behavior of the soil. This was necessary for evaluation of foundation and slope stability. The tests were performed on samples obtained by means of a thin-wall tube sampler. The effective stress envelope was computed at the maximum effective principal stress ratio. Because the theoretical effective cohesion of a nonplastic to slightly plastic silt is approximately zero, the design effective stress cohesion was assumed to be zero. This indicates an effective stress angle of internal friction of 42 degrees for the nonplastic silt and 38 degrees for the slightly plastic silt. The results of the triaxial shear tests are shown in Plates 13 to 16 of Exhibit D.

Unconfined Compression Test

Unconfined compression tests were conducted according to ASTM D2166 to provide strength data for slope stability analysis. These tests were performed on disturbed samples obtained by means of a split-spoon sampler. The maximum value of axial stress from the axial stress versus axial strain curve was taken as the unconfined compressive strength. The undrained shear strength was approximated as one-half of the unconfined compressive strength. The results of these tests are shown in Plate 17 of Exhibit D and in Figures E-2 and E-3 of Exhibit E.

Consolidation Tests

Consolidation tests were performed according to ASTM D2435 on relatively undisturbed samples of the silt layer to provide an estimate of the compressibility of the soil. Samples were obtained by means of a thin-wall tube sampler. The results of these tests are shown in Plates 4 to 12 of Exhibit D.

Compaction Tests

Compaction tests were performed according to ASTM D1557 on samples of borrow materials to determine the maximum dry density and the optimum water content. The results of these tests are shown in Plate 17 of Exhibit D.

Chapter 7
ENVIRONMENTAL CONSIDERATIONS

IDENTIFICATION OF ENVIRONMENTAL CONCERNS

Some of the potential environmental impacts of a dam on the Eagle River were discussed in the MAUS report (1979). These potential impacts include:

- Physical impacts
  - Air pollution
  - Noise Pollution
  - Water quality

- Biological impacts
  - Vegetation
  - Fish
  - Birds
  - Mammals

- Socio-economic impacts
  - Historical and archeological sites
  - Land use
  - Recreation

AWSU requested identification of the environmental concerns that may be related to the development of a water supply reservoir on the Eagle River. To accomplish this, CH2M HILL personnel met with Municipality of Anchorage, State of Alaska, Federal, and Eklutna, Inc., personnel during the week of February 2, 1981. These meetings were held with people from single agencies or small groups of agencies with related interests.

The following agencies and organizations were contacted:

- Federal Agencies

  Department of the Army: Corps of Engineers; Fort Richardson Command, Environmental Office and Utilities Division

  Department of Commerce: National Marine Fisheries Service

  Department of Interior: U.S. Fish and Wildlife Service

  U.S. Environmental Protection Agency

- State of Alaska

  Department of Environmental Conservation

  Department of Fish and Game

  Department of Natural Resources: Division of Forest, Land and Water Management; Division of Geological and Geographic Surveys; Division of Parks

- Municipality of Anchorage: Planning Department

- Eklutna, Inc.

We did not attempt to establish priorities for the environmental concerns that were expressed, but we believe that the common expression of a particular concern indicates that the concern is relatively important.

The persons contacted expressed concern over adverse effects on natural resources, on the human environment, and on the visual or aesthetic quality of the area. The most commonly expressed concerns were over effects of the proposed project on fisheries and water quality. Adverse effects on water quality could potentially affect both fisheries and human health.

Our results are not conclusive because some groups, such as representative residents of the Eagle River Valley, were not contacted. Such a group might have serious reservations about the visual impact of the project.

ENVIRONMENTAL CONCERNS

Fisheries

The extent of the fisheries resources in the Eagle River is not well known. The persons contacted indicated that the existing resources should be identified and protected. Several specific issues were raised concerning fisheries resources. Studies would be required to determine the extent of the fisheries resources.

Loss of Habitat

As a result of the proposed project, the known spawning area at the mouth of the South Fork of the Eagle River would be inundated during the winter, probably resulting in the smothering of any eggs deposited there. The main channel of the Eagle River in the reservoir area is not thought to be heavily used by spawning salmon, but some losses may occur there also.

Fish Passage Facilities

Salmon are known to spawn in the North Fork of the Eagle River, a semi-clear-water tributary that occupies a previous channel of the Eagle River and is now fed by springs and small tributaries. Overflow from the main stem of the Eagle River sometimes occurs, feeding sediment-laden water into the North Fork. The adult salmon migrate from Knik Arm through the Eagle River to the North Fork to spawn, and the juvenile salmon migrate from the North Fork to Knik Arm. There was general consensus among the fisheries agencies that maintenance of natural runs was preferable to attempted mitigation by means of hatcheries or planted fish. Fish ladders are preferred over trap-and-haul facilities for moving adult salmon upstream. Juvenile salmon migrate out of the river primarily from mid-May to the end of June. The reservoir would be almost empty during that period and the outlet gates would be open, so juvenile salmon would be able to pass freely downstream. Some young salmon may be lost unless screening is provided at the water supply diversion intake.

Flow Requirements

The MAUS suggested an approximate minimum flow of 31 cfs (20 mgd) for Eagle River below the diversion point (U.S. Army Corps of Engineers, 1979). This value represents the minimum average flow recorded during the relatively short period of record. The extent to which the lower Eagle River is used by salmon and trout is not presently known. If extensive use is found to occur, minimum flows in excess of 20 mgd may be required. Studies are needed to determine the importance of the lower Eagle River for fisheries.

Sediment

Sediment would be deposited in the reservoir; however, because the reservoir would be emptied in the summer, the flushing action of the Eagle River should help mitigate this impact. The proposed project will probably result in a net decrease in sediment discharged by Eagle River downstream of the dam.

The project may also alter the seasonal pattern of sediment discharge. Not all of the sediment in the turbid water stored in the summer is expected to settle in the reservoir. Winter releases from the reservoir are thus expected to contain more suspended fine sediments than the natural winter river flows. The winter releases may also be warmer than the natural winter flows. If extensive spawning occurs in the lower river below the dam, it could be affected by increased discharge of sediment during the winter and the potentially higher water temperature.

Mitigation of Fisheries Losses

If the project results in fisheries losses, mitigation would probably be required. Measures discussed included spawning channels or a hatchery downstream from the reservoir, increased production at existing hatcheries, enhancement and management of known spawning areas in the North Fork of the Eagle River, and enhancement in other watersheds. The agencies, in general, indicated that protection and enhancement of natural populations are best. Eklutna, Inc., suggested that mitigation through hatcheries would be acceptable.

Changes in Microclimate

The water stored in the reservoir would probably cause some localized changes in climate. While the reservoir is operated for winter storage only, winter air temperatures in the immediate vicinity should be somewhat higher. If the reservoir were converted to year-round storage, summer temperatures in the vicinity of the reservoir would be slightly lower. Any small changes in microclimate would probably be restricted to the immediate vicinity of the reservoir.

Wildlife

Primarily, the agencies expressed concern over loss of habitat by wildlife. Specific items of concern are included in the following discussion.

Loss of Habitat for Game Animals

A variety of small game animals and moose are known to use the Eagle River Valley, including the reservoir site. Clearing the reservoir site of vegetation and seasonal filling of the reservoir would result in a net loss of wildlife habitat. Some plants, such as willows and grasses, tolerate periodic inundation. These plants might persist and thrive in the upstream part of the reservoir and around the margins in shallow areas.

Loss of Habitat for Nongame Species

Raptors, including bald and golden eagles, use the Eagle River area. Nesting sites have not been observed in the project area. It is not clear whether the project will have a negative effect on raptors, but the possibility exists. A variety of small birds also nest in the project area. There would be a net loss of habitat for these species and for small mammals living in the area of the reservoir. Shorebirds that feed on mudflats might gain some habitat from the project.

Groundwater

Depending on local soils, the proposed reservoir could affect groundwater resources in the immediate area. A specific concern was expressed that the presence of the reservoir would raise the level of shallow aquifers in the area around the shoreline and downstream from the damsite. It is not known whether such effects might be positive or negative. Private and public wells in the area are dependent on local aquifers.

Water Quality

Concerns were expressed over the effects on the quality of Eagle River water, particularly in the reservoir, caused by discharges from man-made sources in the watershed. Another concern was the effect on water quality, downstream from the project, because of the loss of dilution water for discharges from the Eagle River Sewage Treatment Plant. There were several items of specific concern.

Leachate from Old Dump

A former dump is located on the south side of the Eagle River about 1 mile west of the damsite (Figure 1-3). The ground surface in this area drains to the east through a natural wetland and discharges to the Eagle River immediately upstream from the proposed damsite. Unregulated dumping occurred for a number of years; therefore, there is a possibility that toxic leachate may find its way to the reservoir. A water quality sampling program would be required to determine if leachate is a problem. If toxic leachates are found, diversion and/or treatment facilities may be required, or the dump may have to be removed.

Septic Systems

Much of the development in the Eagle River Valley is presently unsewered. It is not known at present whether faulty septic systems are contributing bacterial contamination to the Eagle River. Studies may be required to determine if sources of bacterial contamination exist.

Possible effects of future residential development along the Eagle River were also of concern. The possible restriction of development is discussed further under Land Use.

Recreational Use of the Watershed

A third potential source of water quality problems may be from recreational use of the watershed. The Eagle River and the North Fork are currently popular areas with canoeists and kayakers. Some recreational fishing also occurs in these areas. The Chugach State Park visitors' area at the end of Eagle River Road is another potential source of contamination.

Large expanses of frozen snow-covered mudflats may be exposed in some winters when inflow is low and releases high. These areas would be likely to attract snowmobile riders during the winter months, as would the reservoir surface if ice conditions permit. Use of snowmobiles in the area would pose a threat of contamination by petroleum products and bacteria.

Land Use

There is general concern, particularly among the planning agencies and the Division of Parks, over the issues of land use and planning in the Eagle River watershed. Specific issues range from the need for a comprehensive land use plan to concerns over dam safety.

Need for Land Use Plan

There is currently no land use plan that has been agreed on by the various groups having ownership in, or management responsibility for, the Eagle River drainage area. Several parties suggested the need for the Municipality of Anchorage to participate in such a plan. The project should conform with existing plans.

Effects of the Proposed Project on Land Use Options

The presence of a water supply reservoir could either restrict or enhance land use options in the watershed. For example, recreational use of the area might be restricted, some residential development might be precluded or discouraged, some existing trails would be flooded, and potential access routes would be affected.

Power Line Crossing

It was noted that an existing power line might require relocation or higher poles.

Dam Safety

Concern was expressed over stability of the dam during seismic events and potential effects of dam failure on structures located downstream. The potential effects of altered groundwater conditions on stability of soils in the alluvial benches downstream from the reservoir were also of concern. Records of existing private and institutional wells might provide clues on this issue.

Aesthetic Effects

There are no known major archeological sites in the proposed reservoir area. Specific concerns are with minor archeological sites, the Iditarod Trail, and the visual impacts of the project.

Historical and Archeological Sites

According to Eklutna, Inc., there are probably no major ancient village sites in the project area. There are certainly remains of a number of temporary campsites whose locations are unknown.

Portions of the historic Iditarod Trail pass through the Eagle River Valley and might be inundated by the proposed reservoir. The precise location of the trail in the area is not known, but it may have passed through the present Eagle Heights subdivision, which is near the proposed reservoir site.

Visual Impacts

The conversion of a vegetated river valley into a periodically flooded reservoir would create a visual impact that would be judged as an adverse effect by some residents of the area. The magnitude of this problem cannot be assessed currently because the group most likely to be concerned with visual impacts was not included in the conferences.

SUMMARY

There are a number of environmental concerns that must be addressed in the final design of the Eagle River dam. At present, adequate data are lacking to gauge the potential impacts discussed in this chapter.

Studies must be made prior to or as part of the final design of Eagle River Dam to obtain the following data:

- Fisheries. Insufficient data are available about the type, number, migration, and distribution of fish in the river. These data are essential to evaluate potential impacts of a dam and to provide satisfying mitigation of these impacts.

- Sediment. Insufficient data are available at present to confidently predict the potential impacts of a dam and reservoir on sediment deposition and transportation patterns.

- Water quality. No data are available on the effects of the old Eagle River dump on surface and groundwater quality. Soils data must be obtained to evaluate the need for diversion, treatment, or other measures, if any, that may be required to develop the Eagle River as a water source.

Because of the large size of the proposed project, the number of potential impacts, the current insufficient amount of data available, and the potential magnitude of some of these impacts, it is expected that an environmental impact statement (EIS) would be required. We anticipate that preparation of an EIS for this project may require 3 or more years. If the Eagle River dam is to be further considered, determination of the need for an EIS should be made as soon as possible. If a need is established, studies to develop the EIS should then be started.

Chapter 8
MAJOR PROJECT ELEMENTS

An analysis was conducted to establish the feasibility of constructing a dam on the Eagle River and to develop the basic concepts and geometry for such a structure. The analysis of the conceptual design of the dam and reservoir is based on data established by the field and laboratory investigations described in previous chapters. This chapter describes the major elements of the project.

The Eagle River dam would consist of an 80-foot-high compacted earthfill dam with a 100-foot-wide gated chute spillway on the right abutment. The spillway gates would be 30-foot-square radial gates. The spillway would discharge into a horizontal apron hydraulic jump stilling basin. The dam would have two low-level outlet conduits, each 10 feet square. Each conduit would be controlled by a roller gate located in a control tower near the upstream end of the conduits. The base of the control tower would be buried in the upstream shell of the dam and would extend above the reservoir surface. Access to the control tower would be provided by a bridge from the dam crest. Fish facilities would be located near the stilling basin and on the upstream face of the dam. The location of the intake structure for the transmission pipeline has been assumed to be in the reservoir and separate from the dam, as shown in the MAUS report (U.S. Army Corps of Engineers, 1979). A plan of the reservoir is presented on Figure 1-2. A site plan showing the dam and major project elements is shown on Figure 8-1.

FOUNDATION AND ABUTMENTS

The dam would be founded on a combination of dense glacial fill and outwash gravels (left abutment), stiff lacustrine deposits (central portion), and alluvial channel gravels (spillway and right abutment). The site would be prepared by digging a foundation excavation in the area to be occupied by embankments or structures. All overlying organic material, topsoil, recent or loose alluvial sediments, and unconsolidated colluvial material would be removed from this area. All materials that might be compressible, subject to liquefaction during seismic loading, subject to erosion piping, or produce a zone of weakness or high permeability would also be removed. This removal would provide the best possible bond between the dam and its foundation.

A core trench, as shown on Figure 8-2, should be excavated for the entire length of the dam. The excavation should be to a depth sufficient to ensure that loose or disturbed material beneath the core is removed or treated to prevent liquefaction or seepage concentrations beneath the core. It is anticipated that the core trench would typically extend about 5 feet into firm, undisturbed glacial or alluvial deposits. The total depth below existing ground may be 20 feet or more in some areas. The actual depth should be field-determined by an engineering geologist or geotechnical engineer during dam construction.

Organic material removed from the foundation excavation should not be placed in the dam embankment. Inorganic material removed from the foundation and core trench excavations can probably be placed in the core or the shells of the dam, depending on its gradation and moisture content. The suitability of material for placement in the dam must be field-determined at the time of excavation. Undesirable materials should be wasted.

Dewatering

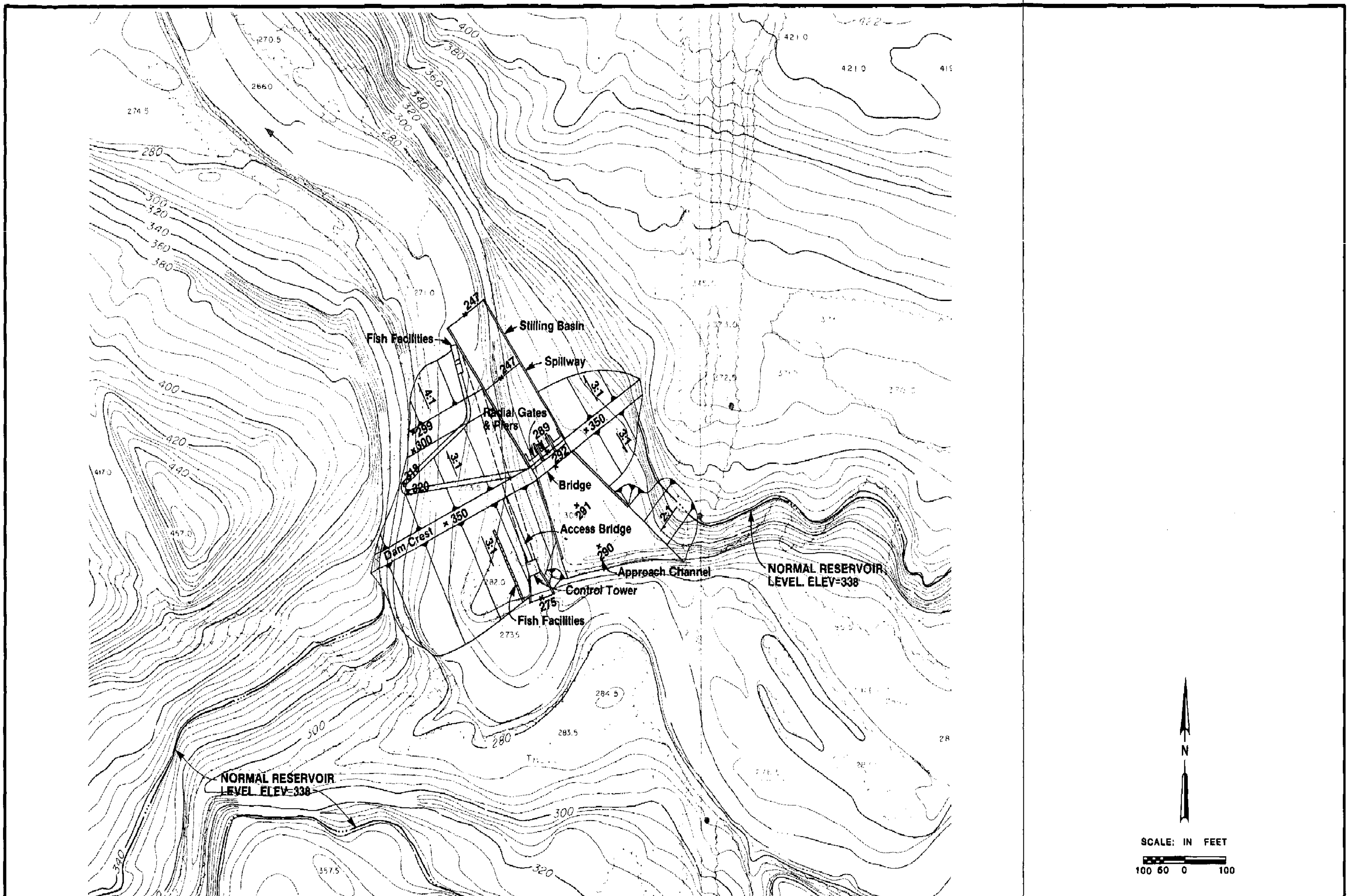
Dewatering would be necessary during construction to control river underflow and to prevent other groundwater from entering the excavations. The quantity of water would vary with location and weather. It appears unlikely that pumping from the excavations, by itself, would be adequate to control groundwater. Studies should be made during design to determine the advisability of wellpoint, slurry trench, deep-well, or other dewatering systems.

An artesian flow of 1 to 2 gallons per minute was observed during drilling at boring B-6 (see Figure 5-1). According to this boring, the source of this water is a gravel layer found just above the rock at the bottom of the boring. The top of this gravel layer was observed in boring B-6 at an approximate elevation of 230 feet. The lowest point in the core trench excavation is presently estimated to be at about elevation 255 feet. It may be necessary to dewater this gravel layer using deep wells and pumping to prevent heaving of the bottom of the core trench excavation.

Surface water that flows into the foundation and core trench excavations and groundwater not intercepted by the construction dewatering system should be collected in ditches, led to sumps, and pumped from the excavation. Control of water is of critical importance because of the moisture sensitivity of the lacustrine deposits beneath the damsite.

Foundation Preparation

Special preparation of the core trench excavation is often required to ensure a firm bond between the core trench material and the dam foundation and to prevent seepage concentrations adjacent to the core trench fill. The exact type of preparation cannot be determined with certainty until the core trench excavation is open. It is anticipated that the only preparation necessary would be removal of all loose material and removal of any gravel stringers or other pervious zones passing beneath the core



Elevations are in Feet

Figure 8-1
Major Project Elements

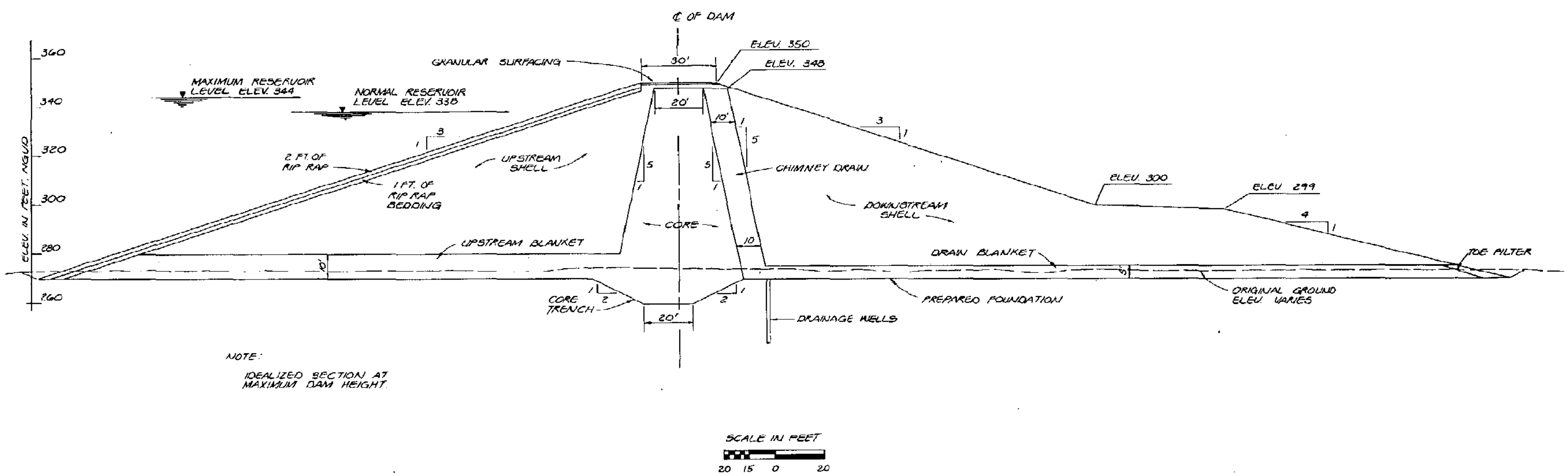


Figure 8-2
Dam Section

of the dam. The base of the core trench should then be proof-rolled to ensure that all loose material has been removed. Care would have to be exercised to protect the foundation material where the foundation is excavated to lacustrine deposits. These deposits are moisture-sensitive, and can be difficult to keep in a dense state if moisture and drainage are not carefully controlled. Other special provisions may be required depending on the conditions disclosed during core trench excavation.

DAM SECTION

The proposed dam section is shown on Figure 8-2 and the profile is shown on Figure 8-3. The dam would be a zoned earthfill dam with a thick central core, an upstream blanket, a chimney drain, and a downstream drainage blanket. The upstream and downstream shells would be constructed of well-graded sands and gravels excavated from the reservoir area and spillway. The chimney drain and downstream drainage blanket would be constructed of selected processed material from the shell borrow area or from river bars in the reservoir. The core would be constructed of low permeability sandy silt excavated from the reservoir area. During final design, more detailed borrow area explorations should be conducted to verify that sufficient material is available to construct the dam and to attempt to locate silty material for the dam core that has a significant fraction of sand and gravel. This would reduce its susceptibility to internal erosion (piping).

The dam shells, the chimney drain, and the downstream drainage blanket would be well compacted to minimize the possibility of liquefaction during seismic shaking.

The core and the upstream blanket also would be compacted to a high degree of density. In addition, they should be compacted at a moisture content near or wetter than optimum moisture content. This would reduce the core material's permeability and increase its flexibility.

Riprap should be placed on the upstream face of the dam to protect the dam from wave damage. The riprap would be well-graded, imported stone with a maximum size of 18 inches, a median size of 12 inches, and a minimum size of 6 inches. This material would have to be imported because a suitable source of riprap is not available at or near the damsite.

Bedding should be provided to act as a transition between the riprap and the upstream shell material. The bedding would prevent embankment material from washing out through the riprap because of wave action. Riprap bedding would be a well-graded material with a 6-inch maximum size. It is probable that suitable material can be selectively excavated or processed from the borrow pit for shell material. If not, imported material would have to be used.

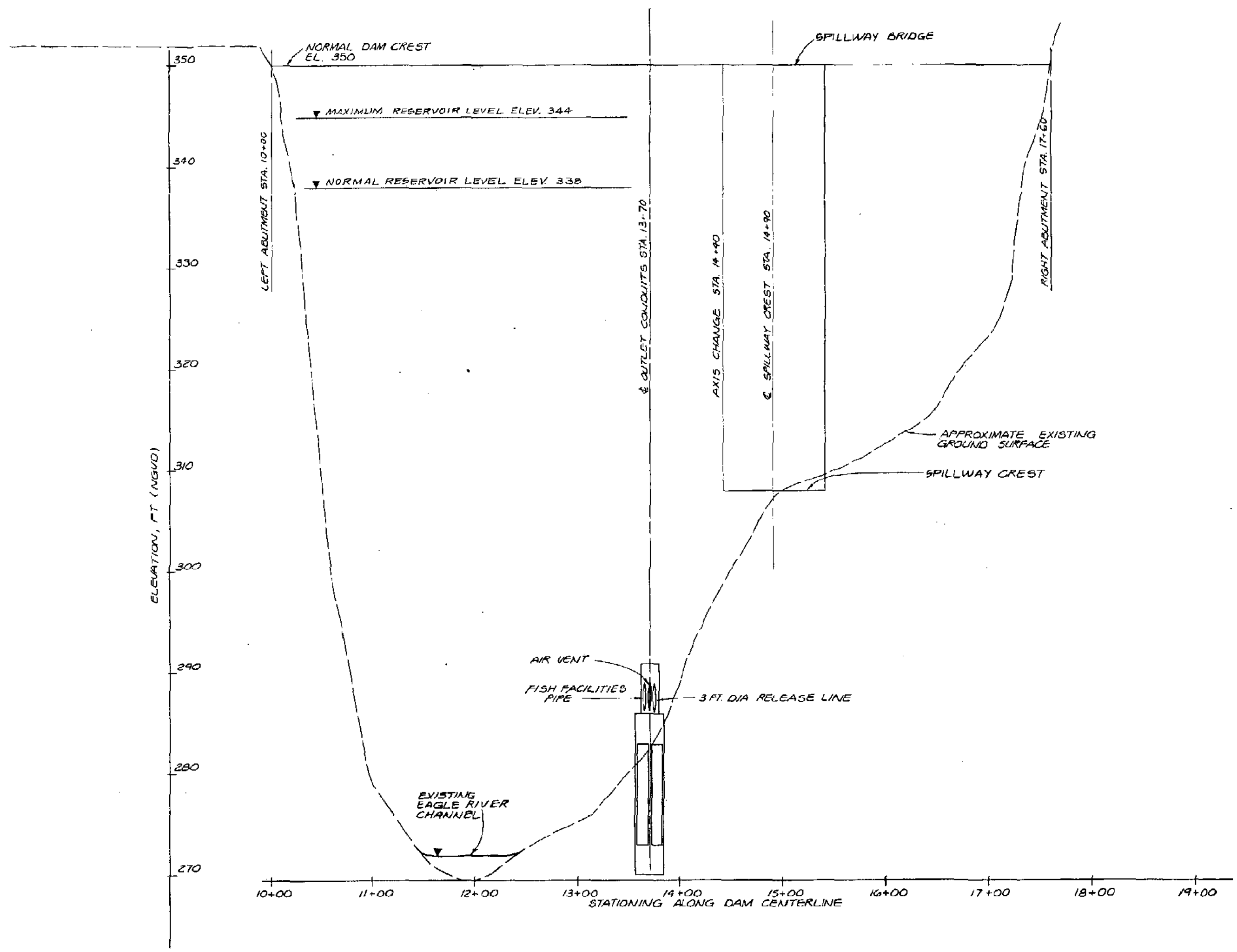


Figure 8-3
Dam Profile

The downstream face of the dam would be covered with topsoil and planted to minimize erosion of embankment material. The topsoil would be selected from material stripped for foundation excavation.

Gravel surfacing would be used on the dam crest to prevent surface erosion and to allow its use as an access roadway. Gravel surfacing would be a well-graded crushed stone with a maximum size of 3/4 inch.

Seepage

Water would leak from the reservoir and dam. Seepage can occur through the dam core, under the dam core, through the dam abutments, and through the reservoir sides and bottom. There are two principal adverse effects of seepage: water loss and reduced dam stability. Because minimum streamflow must be maintained on the Eagle River, some water loss is acceptable as long as it does not reduce the dam's stability.

The presence of the lacustrine silt deposit at a depth of 10 to 20 feet below the riverbed is expected to minimize the amount of seepage passing beneath the dam. Most of the seepage is expected to pass through the sand and gravel layers located above the silt in the dam abutments. Based on the results of the laboratory gradation tests, we expect the permeability of the abutment materials to vary, with an average permeability on the order of 10^{-4} feet per minute.

The estimated seepage loss through the dam foundation and abutments is not expected to exceed 2 cfs (1.3 mgd). It is anticipated that most of this water would discharge into the Eagle River immediately downstream of the dam. This seepage would contribute to the minimum flow of the river.

One of the potential adverse effects of seepage at the site is erosion of the steep slope immediately downstream of the dam on the southwest valley wall. This could result from leakage through the left abutment. The surface material in this area consists of unvegetated silty sand and gravel. Frost action will assist in the erosion process.

To minimize the seepage that may emerge in this area, the natural soil cover on the upstream slope of the left abutment should not be disturbed during construction. This would help provide a natural low permeability blanket to reduce the quantity of seepage entering the left abutment. Additionally, silt from the reservoir water would be drawn into any areas of infiltration on the left abutment during the first few years of reservoir operation. This silt would help form a natural blanket over seepage infiltration points on the left abutment and over the entire reservoir area.

If seepage emerging on the valley wall continues to cause erosion after several years of dam operation, horizontal drains could be installed in this area. Because of the potential difficulties associated with keeping these drains clear of ice, they should be considered only if the other methods mentioned above fail to control the erosion satisfactorily.

A blanket of core material beneath the upstream dam shell would be provided. This would increase the length of the flow path for seepage passing beneath the dam, and thus reduce its quantity. It also will reduce the magnitude of the seepage forces beneath the central portion of the dam and would increase the dam's stability.

To further reduce seepage pressures beneath the dam, drain wells will be provided downstream of dam core. These are shown on Figure 8-2. These drain wells would collect the seepage and discharge it at controlled locations rather than permit it to exit uncontrolled where the dam meets the abutments.

Seepage through the reservoir sides and bottom can only be roughly estimated. The Eagle River Valley contains a very heterogeneous assemblage of glacial and alluvial deposits overlaying relatively impervious bedrock. Seepage that flows into the sedimentary materials in the valley would, for the most part, not be lost to the underlying bedrock. However, the seepage path cannot be evaluated, except on a local scale, near the damsite. The presence of widespread glacial lake deposits beneath the damsite and in the reservoir area would help reduce the seepage losses. While it is extremely difficult to estimate the amount of potential seepage, we anticipate that it would probably not exceed 4 cfs (2.6 mgd). This estimate is based on the general geology of the valley, the results of Task 1 to date, and the gradation characteristics of the materials sampled during the damsite exploration.

Cracks may develop in dams from differential movements or from irregularities in the foundation materials. Seepage in open cracks can cause enlargement because of flow concentration along the cracks. This process, referred to as "piping," can lead to dam failure.

In order to provide adequate protection against failure of the dam by piping, a chimney drain immediately downstream of the core has been provided in the design. This drain would prevent migration of fine material from the core. It would also provide drainage for the downstream shell of the dam. Seepage intercepted by the chimney drain would be conveyed to the downstream toe of the dam through the downstream blanket and would be discharged into the Eagle River from the blanket. A graded filter is provided at the downstream end of the blanket to prevent loss of fines into the river. Freezing of the discharged seepage is not anticipated to be a problem. The seepage water temperature would be above freezing, and would discharge continuously below river level at the toe of the dam.

Settlement

Some settlement of the dam is expected. Short-term settlement would result from elastic compression of the foundation and embankment materials during the construction of the dam.

Long-term settlement would result from consolidation of compressible lacustrine silt deposit in the dam foundation. The geologic evidence and the results of the laboratory consolidation tests indicate that the amount of settlement would be tolerable because the silt deposit has been subjected to a high preconsolidation pressure in the past. The amount of potential settlement is dependent on the slope of the reloading curve for the stiff silt; the large size of the dam and the resulting great depth of its influence on in-situ stresses; and the thickness of the compressible soil. The methods presently available to estimate the amount of consolidation of highly preconsolidated soils are expected to yield only an approximation of the ultimate settlement. The settlement caused by consolidation of the compressible layers in the dam's foundation is expected to be on the order of 6 inches. To compensate for this settlement, the dam crest would be constructed to an elevation above the nominal crest elevation by an amount varying from 1/2 foot at the dam ends to 1 foot at the point of maximum height.

The time over which the settlement would occur is dependent on the permeability of the soil and the location of more pervious drainage layers. Only an approximate time can be estimated, because of the complexity of the drainage conditions beneath the dam. We anticipate that 90 percent of the settlement will occur within approximately 1 year after the completion of construction.

Stability Analysis

The stability of the dam was evaluated with respect to resistance of the embankment to shear failure under different loading conditions. A generalized section of the embankment at its maximum height is shown on Figure 8-2. This generalized section was analyzed for stability under the following conditions:

- Downstream slope, normal operating conditions
- Upstream slope, normal operating conditions
- Upstream slope, sudden complete drawdown conditions

Stability of the dam during earthquake shaking is discussed later under Seismic Considerations. A separate analysis was not made for the end-of-construction conditions because these conditions were assumed to be similar to the sudden complete drawdown conditions.

The embankment consists of a number of zones. To analyze the stability of the embankment, the shear strength parameters were identified for the materials in each of these zones. For the dam shells and foundation materials, the shear strength parameters are based on results of laboratory tests performed on samples of material from the field exploration program. For the dam core, chimney drain, and drainage blanket materials, conservative parameters are assumed.

Table 8-1 presents the shear strength parameters and the unit weights used in the analysis.

Table 8-1
MATERIAL PROPERTIES

Material Type	Shear Strength Parameters: Effective Friction Angle (degrees)	Shear Strength Parameters: Effective Cohesion (psf)	Moist Unit Weight (pcf)
Dam Core and Upstream Blanket	30	0	100
Dam Shells	40	0	140
Chimney and Blanket Drain	40	0	140
Lacustrine Deposits in Foundation	35	0	124
Gravelly Sand in Foundation	44	0	140
Gravel in Foundation	40	0	140

Initially, the stability of the dam under normal operating conditions was analyzed using the infinite slope method for both the upstream and downstream slopes (Taylor, 1948). The results of this analysis indicate that both the upstream and downstream slopes would be stable at a slope of 1.8 (horizontal) to 1 (vertical) with a factor of safety of 1.5. However, as discussed later in this chapter, under Seismic Considerations, it is recommended that the slopes be constructed at a slope not greater than 3 to 1 to reduce the potential for crest settlement from earthquake loading.

After acceptable slopes for the dam shells were determined, it was necessary to analyze potential deep shear surfaces passing through the dam's core and foundation. To do this, more detailed stability analyses were made with STABL, a computer program for analyzing general two-dimensional slope stability problems by a limiting equilibrium method (Siegel, 1975). STABL uses the Modified Janbu method of slices. This method allows for analysis of irregularly shaped shear surfaces as well as circular surfaces. Values of the factor of safety computed by this method are slightly conservative when compared with other equilibrium methods of slices.

The computed factors of safety for the cases analyzed, critical shear surfaces, and the assumed piezometric surfaces are shown on Figure 8-4. In addition, the following assumptions were made:

- Full drainage is expected in the embankment. Full drainage assumes that internal drainage would remove all seepage through the core and the foundation without pore pressure buildup in the downstream shell.

- The drawdown analysis assumed no drainage whatsoever in the core of the dam and complete drainage in the upstream shell. Drawdown was assumed to occur instantaneously from the normal reservoir elevation of 338 feet to the lowest outlet elevation in the reservoir, elevation 275 feet. The riprap and riprap bedding on the upstream face were assumed to drain instantaneously.

Freeboard

Freeboard must be provided to compensate for waves, wave runup on the face of the dam, wind setup on the reservoir surface, possible dam settlement due to earthquakes, reservoir surface perturbations during earthquakes, and a minimal residual freeboard amount.

For the Eagle River reservoir, we estimate that the maximum wave height will be approximately 4 feet. This wave height is based on an effective fetch of 2 miles and a wind speed of 70 mph blowing along the reservoir toward the dam. Under these conditions, 4-foot waves would develop after approximately 20 minutes of sustained wind.

When the waves strike the embankment they would run up on the sloping riprap surface. The estimated height of the runup is approximately 1/2 the wave height, or 2 feet for the maximum expected wave.

One foot of freeboard has been provided to compensate for settlement of the embankment crest during seismic shaking.

RESULTS OF STABILITY ANALYSIS

CASE	CONDITION	COMPUTED FACTOR OF SAFETY	MINIMUM ACCEPTABLE FACTOR OF SAFETY
1	DOWNSTREAM NORMAL OPERATING CONDITIONS	2.4	1.5
2	UPSTREAM NORMAL OPERATING CONDITIONS	2.2	1.5
3	UPSTREAM INSTANTANEOUS COMPLETE DRAWDOWN	2.0	1.0

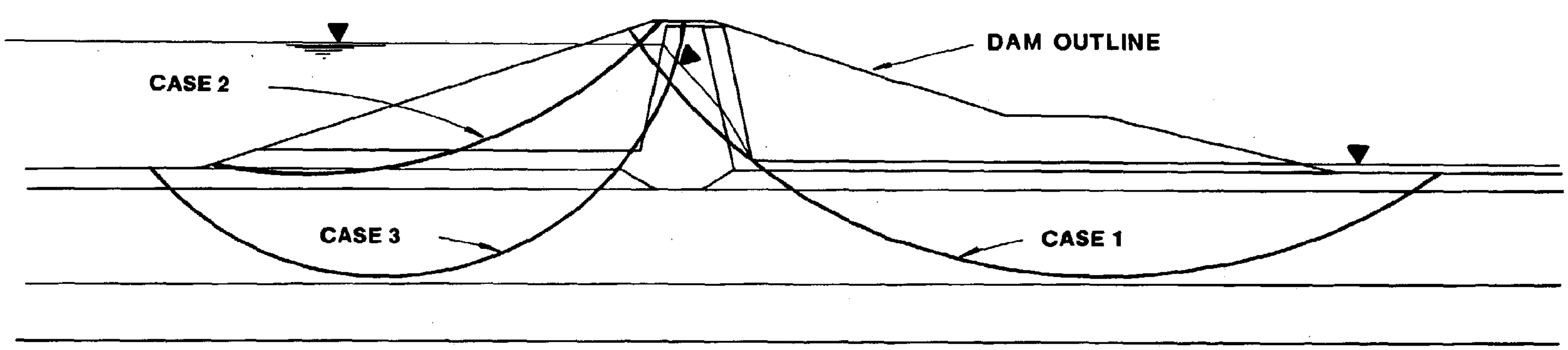


Figure 8-4
Stability Analysis

Five feet of residual freeboard have been provided to compensate for other earthquake-related unknowns such as potentially greater crest settlement during unexpectedly large post-construction embankment or foundation settlements, unusally large waves, or seiches or other reservoir surface perturbations.

This provides a total of 12 feet of freeboard above the normal reservoir level. During the PMF (refer to Chapter 3), 5.5 feet of freeboard would remain above the maximum reservoir level.

Seismic Considerations

A preliminary estimate of the expected peak and effective bedrock accelerations during the maximum credible earthquake (MCE) was made by Lindval, Richter & Associates. Their report is included as Exhibit C. They estimate that peak bedrock accelerations of 0.40g may occur during the MCE. They also have estimated that the expected maximum effective bedrock acceleration would be approximately 0.33g. Accelerations of this magnitude could cause embankment failure if accelerations were amplified by soil overlaying bedrock or by soil in the embankment. However, as discussed previously, the foundation soil is extremely dense, and the embankment soil would be compacted to a high degree of density during construction. Therefore, these materials should not amplify bedrock motions significantly.

The peak acceleration is repeated only a few times during the MCE. Newmark (1965) provides a method to estimate slope movements resulting from earthquake shaking. Using a horizontal peak acceleration of 0.5g, Newmark's method predicts a down-slope movement of 0.3 foot. This slope movement takes place along a 3 to 1 slope, and would thus correspond to approximately 0.1 foot of vertical displacement. The expected deformation from peak acceleration is small and would probably only cause minor localized movement of the embankment.

Several features in the conceptual design of the dam section provide additional measures of safety against dam failure during an earthquake. The materials proposed for construction of the shells and drainage zones are expected to have negligible unconfined compressive strength. This would prevent these materials from sustaining an open crack without collapsing. Therefore, after any displacement caused by earthquake shaking, even if a large open crack existed in the core of the dam, there could not be an open crack through the shells or drainage zones. The chimney drain immediately downstream of the core would be cohesionless and would thus be capable of healing itself of any cracks that may be caused by an earthquake or from settlement. This would keep the drain continuous and provide a filter to hold the core material in place, preventing dam failure by erosional enlargement of an open crack. The cohesionless nature of the upstream shell would provide material that can wash into any crack that may open in the dam core.

Because of its well-graded nature, the proposed shell material is expected to have relatively low permeability. Therefore, the amount of water that could leak through a crack in the core would be limited by the permeability of the upstream shell. The gravel in the shell and drain materials, and the fact that these materials are well graded, makes them inherently stable against internal erosion (piping). The worst condition that would exist after a major earthquake would be a zone of loosened material in the dam shells and the filter zones.

Although the embankments might be severely damaged by the MCE, catastrophic failure should not occur.

SPILLWAY

The spillway would be located on the right side of the dam, with dam embankment on both sides of the spillway (Figure 8-1). It would consist of an excavated approach channel, a concrete ogee crest topped by radial gates, and a 100-foot-wide concrete chute leading to the stilling basin at the downstream toe of the dam. The spillway has been designed to pass the PMF with approximately 6.5 feet of surcharge above normal reservoir level. A profile of the spillway is shown on Figure 8-5.

The approach channel to the spillway crest would be unlined, 250 feet long, tapering from 200 feet wide at the entrance to 100 feet wide at the upstream face of the spillway crest. The approach channel would be at elevation 292 feet at the upstream face of the spillway. It would be excavated on an adverse slope of 0.01 foot per foot to allow for drainage when the reservoir is lowered in the summer. This excavation would improve hydraulic operating characteristics of the spillway. In addition, the excavated material could easily be used in the dam embankment. Concrete retaining walls (wing walls) would be provided where the channel penetrates the dam embankment.

The spillway crest would be placed at elevation 308 feet. The crest would be ogee-shaped with a 30-foot design head and a vertical upstream face. The crest would be divided into three 30-foot-wide bays, each containing a radial gate with a 30-foot damming height. Two piers and the side walls of the spillway chute would be used to support the gates, which would be operated remotely from the proposed Eagle River water treatment plant. Hoist mechanisms are located on the deck, 42 feet above spillway crest.

The radial gates would be provided with flexible seals at the sides and bottom of the gate skin plate. These seals would contact side rubbing plates and a sill plate embedded in the spillway structure. These seal plates should be heated in the winter to ensure that the gates can be opened in the event of an early spring flood occurring after a rapid rise in temperature. Heating

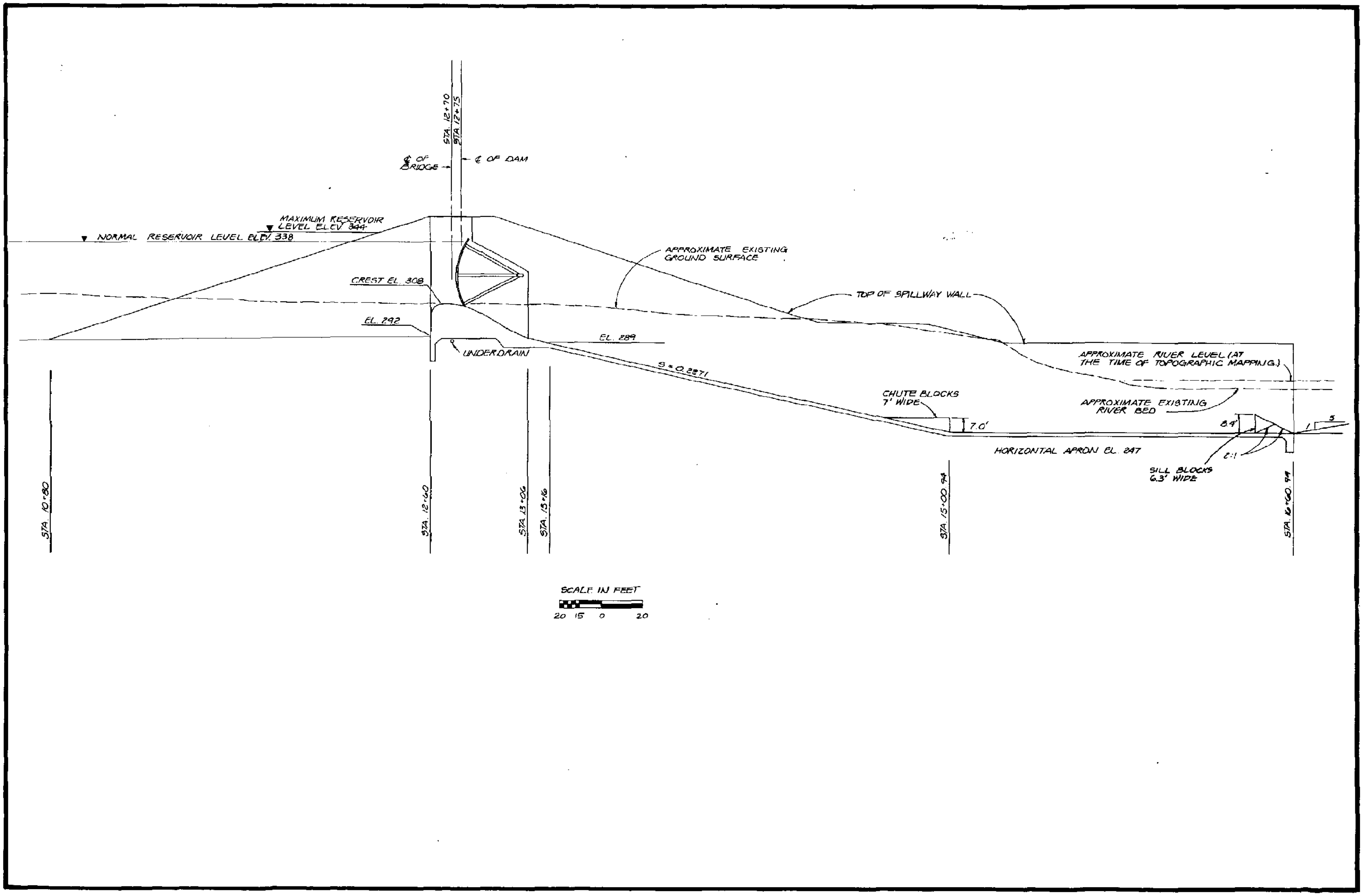


Figure 8-5
Spillway Profile

of the seals would also prevent tearing or other damage to the seals that could occur if ice were allowed to form around them or on the seal plates.

The seal plates would be heated by circulating warmed oil or antifreeze solution through pipes embedded behind the seal plates. The seal plates need only to be kept warm enough to prevent ice formation.

A 60-foot-radius reverse curve would be used as a transition between the ogee crest and the 100-foot-wide chute. The curve would become tangent to the chute at approximately elevation 289 feet and continue on a constant slope of 0.227 foot per foot for 185 feet to the stilling basin. Some cost savings may be realized by tapering the spillway chute; this detail can be considered in the final design.

The spillway is sized to pass 57,000 cfs with the reservoir at normal reservoir level, elevation 338 feet. This provides capacity to handle a flood equal to one-half of the PMF without surcharging the reservoir. The spillway rating curve is provided in Figure 3-30.

The above conceptual design differs considerably from that conceived in the MAUS (U.S. Army Corps of Engineers, 1979). The Corps' design assumed a reservoir with a maximum surface elevation of 325 feet. The spillway was to have two radial gates, each 50 feet wide by 50 feet high. The floor of the spillway was to be level with the natural riverbed (approximately at elevation 268 feet) to allow nearly unrestricted passage of summer flows. This concept was not used for the present study because our reservoir operations study indicated that the normal reservoir level needs to be at elevation 338 to meet the required water demand. The 70-foot-high gates required to store water to this elevation were considered to be uneconomical.

Several other spillway configurations were considered for the dam: uncontrolled spillways, wider spillways, and spillways with no surcharge storage. The PMF was routed through the reservoir using uncontrolled, ungated spillways of varying widths. Surcharge above the normal reservoir level varied from 29 feet for a 100-foot-wide ogee spillway to 19 feet for a 250-foot-wide ogee spillway. However, a gated spillway is necessary to minimize the flooding of the Eklutna, Inc., land upstream of the proposed reservoir site. Also, space limitations at the site indicated that construction of a spillway much wider than 100 feet would become very difficult and expensive. Routing studies indicated that a 160-foot-wide spillway with 30-foot-high gates would be required to pass the PMF without surcharging the reservoir above normal reservoir elevation 338 feet. The expense of this structure was considered excessive for the benefit of no surcharge storage.

The proposed 100-foot-wide gated spillway provides no surcharge storage up to one-half the PMF, while allowing a reasonable 6.5-foot surcharge during the PMF.

STILLING BASIN

The stilling basin is located at the toe of the dam. Space limitations at the site make it necessary to combine the energy dissipation structures for the spillway and outlet conduits into a single stilling basin. This is best accomplished at this site with a horizontal apron hydraulic jump basin. A standard Basin II, as shown in the U.S. Bureau of Reclamation's (USBR) Design of Small Dams (1974) was chosen for use. The basin is 100 feet wide, 160 feet long, and has a floor elevation of 247 feet. It was chosen because the incoming flow velocities can exceed 60 feet per second. This design does not incorporate baffle piers on the basin floor because of the potential for damage by the high velocity flow. However, the basin does have a row of chute blocks at the intersection of the chute and the basin floor and a dentated end sill to help shorten the hydraulic jump length. The exit channel is excavated on a slope of 20 percent until it meets the streambed level.

The outlet conduits empty into the spillway stilling basin by penetrating the left stilling basin wall. The outlet conduits would normally be discharging only when the spillway is not operating. Under this condition, the water level in the stilling basin would be very near the invert of the conduits, and the stilling basin would function as a plunge pool.

Space limitations at the site and high construction costs led to the stilling basin being designed for a spillway flow of one-half the PMF. The one-half PMF is an extremely infrequent event, with a very low probability of occurrence during the life of the project. However, because the spillway design calls for passing the full routed PMF, there is some probability that the capacity of the stilling basin would be exceeded. Under these conditions the stilling basin may be damaged, but the integrity of the dam is expected to remain. The possibility of having to perform some repairs on the stilling basin is considered to be an acceptable risk in view of the extremely low probability of such an event occurring.

OUTLET WORKS

Outlet conduits would be provided to divert the river during construction, to control downstream flows, to assist in control of the reservoir surface level, and to provide a conduit for possible future hydroelectric facilities. They would be located near the spillway and discharged into the stilling basin.

Several configurations, sizes, and number of outlet works were considered before the proposed design was chosen. One consideration was to have the outlet conduits within the spillway structure rather than in the spillway stilling basin. This would have allowed a permanent pool 35 to 40 feet deep, with the following advantages:

- Less pumping would be required to transmit water in the pipeline
- Minimum streamflow discharges would have enough head to be used for hydroelectric generation
- All low-level dam penetrations and the associated control tower would be eliminated
- All spillway and outlet controls would be close together
- A fish ladder could be more easily provided

There would be the following disadvantages to this alternative:

- Additional dead storage would be created because the area below the outlet works would not be usable and would fill up with sediments
- Construction of the dam would be more complicated because the contractor would have to build the spillway first, then build temporary structures in the river to divert the river over the spillway while the dam is built
- The spillway section would be much more complicated with penetrations for the outlets, gate chambers, passageways, and control rooms

It should be noted that the first disadvantage is not major because this dead storage can be recovered with only a minimally higher dam and reservoir. The advantage of increasing the head could be achieved by throttling the low-level outlets with the roller gates.

The proposed design of the outlet works would consist of two 10-foot-square reinforced concrete conduits and a 3-foot-diameter pipe. Flow in the square conduits would be controlled by roller gates.

The conduits would have an entrance invert elevation of 275 feet, 3 feet above the existing river bank. They would have a slope of 1 percent and have an exit invert at 270.5 feet at the stilling basin wall. Figure 3-31 shows the rating curve for one conduit. When both conduits are flowing, the combined flow would be approximately twice the value shown on the rating curve.

The outlets would have trashracks and stoplog slots at the entrance. Downstream of the entrance a wet well is provided for the roller gates. Another set of stoplog slots is provided at the downstream end of the conduits. Stoplogs can be used to dewater the conduits for inspections, repairs, and modifications. The conduits enter the stilling basin through the side wall at about a 15-degree horizontal angle from the spillway axis. Therefore, one of the square conduits would be longer than the other. The average conduit length is 450 feet.

The 3-foot-diameter pipe provides the minimum downstream flow requirements when the roller gates are closed. A valve is provided for accurate control of the flow volume in the pipe. Figure 3-32 shows the rating curve for this pipe and for two larger pipes. A larger diameter pipe may be required if: (1) final design fish flow requirements are higher than the 31 cfs used in this analysis, or (2) the 108-cfs municipal demand is transmitted through the dam to a downstream pump station. The MAUS report (U.S. Army Corps of Engineers, 1979) suggested withdrawing water from the reservoir through a pump station located in the reservoir. During final design, consideration should be given to alternative withdrawal points, including immediately downstream of the dam or along the river near or downstream of the Glenn Highway bridges.

The square conduits would be designed to allow the river to flow very close to natural conditions in late spring and summer. During this time period, the water surface at the upstream side of the dam would not be higher than elevation 290, except for very short periods. This design would promote the movement of sediments through the empty reservoir during the period when most of the sediment is being transported.

The final design of the outlet works and pump station intake would determine the number, size, and location of all dam penetrations.

RESERVOIR

The normal pool surface would be at elevation 338 feet, with a reservoir surface area of 2,530 acres and a total storage volume of approximately 55,000 acre-feet. The maximum pool surface would be at about elevation 344.5 feet, with a reservoir area of approximately 2,840 acres, and a total storage volume of approximately 71,200 acre-feet.

A plan view of the reservoir area is presented on Figure 1-3.

Land Considerations

At present, the site for the proposed reservoir is covered by scattered spruce, birch, and other trees. Unless the fisheries agencies request that the dead trees be left to provide habitat for fish, they should be cleared and removed from the reservoir prior to the first filling. Clearing should be accomplished during February through April when the trees are dry and leafless. Stumps and roots should be left in place during clearing to help hold the soil in place and to reduce surface erosion.

During and after clearing, the portion of the reservoir site upstream of the left abutment should be evaluated with additional subsurface exploration and geological mapping in more detail to verify the size and location of pervious sand or gravel deposits. If pervious layers are found to extend through the left abutment, it may be necessary to construct an artifical blanket in this area to reduce seepage losses.

It is preferable to leave the topsoil and the overlying organic mat in place to minimize erosion. Usually, such materials can be left in place without significant impact on water treatability. The possible need to remove the organic mat should be evaluated during final design.

There is a potential for reservoir contamination from leachate from the old Eagle River dump, located west of the damsite. A surface water and groundwater monitoring program will be necessary to evaluate the possible effects of this dumpsite on the proposed Eagle River Reservoir.

Landslides in the reservoir area could occur as a result of sudden drawdown or as a result of rapid saturation caused by water infiltration into the ground surface once the surface vegetation is removed. The normal rate of drawdown should not exceed the withdrawal rate necessary to maintain minimum streamflows and to supply the water treatment plant. Based on the material types reported in previous geologic investigations and on our observations of materials near the damsite, only minor erosional or stability problems of the shoreline, such as shallow surface slides, are expected during normal reservoir operation. However, more detailed studies should be made during design to determine acceptable maximum drawdown rates.

Seismic Considerations

Seismic tremors caused by the filling of the reservoir are not expected because of the relatively shallow reservoir depth and the high degree of preconsolidation of the soils at the site. Several hundred feet of ice and glacial materials have been present over the reservoir area in the past. Thus, the addition of an average of 22 feet of water over the reservoir area is not likely to trigger an earthquake. While a few instances of reservoir-induced seismicity have been postulated, very little information is available on this subject.

During severe earthquakes, incidents of vertical and/or lateral movements of massive earth or rock blocks have been recorded at various localities. Very large areas have been raised, lowered, and/or tilted. Major vertical movements of portions of the reservoir, or massive tilting of the entire reservoir area, could cause the dam to be overtopped. No method is available to predict the possibility of such an occurrence. The only precaution is to provide a reasonably large amount of freeboard.

Waves in the reservoir may be caused by certain types of earthquake motion that produce a slow rocking motion of the entire mass of water in the reservoir. These oscillations are termed seiches. Overtopping may result, although the only instance of record was also associated with massive ground tilting. No seiches caused by shaking alone have been recorded with heights of more than a few feet. Although at the present time no reliable method is available to predict the maximum wave height due to seismic activity, the minimum residual freeboard provides some protection against seiches.

Chapter 9
CONSTRUCTION

AREAS OF UNCERTAINTY

The construction of major earth structures involves more uncertainties than most kinds of construction work. Design considerations for major earth structures must be continually reevaluated throughout the construction period. During construction, the true character of the subsurface conditions is often found to differ from those indicated by the borings, test pits, and laboratory tests despite thorough, careful investigations prior to construction. If conditions encountered during construction are different from those assumed during design, changes may be required. For this reason the design of a major earth embankment is a continuing process extending throughout the construction period and into the early phases of operation.

The impacts that can occur as a result of the inherent uncertainties related to subsurface conditions can be minimized if:

- The owner recognizes that additional money over and above the initial contract bid amount may be spent during construction

- The designer's representative is on-site to quickly identify "changed conditions," and the appropriate design changes are promptly made by the designer

- Good communications are maintained between the owner, designer, and regulatory agencies so that all responsible parties can approve design changes quickly

The alternative to making design changes during construction is to design the dam so conservatively that it will be adequate under all of the "worst-case" assumptions. This requires a design that may be prohibitively expensive, and wastes money if the worst-case conditions do not actually exist.

The following site conditions present difficulty for embankment construction and add to uncertainty at the Eagle River damsite:

- The main dam requires a large volume of fill that must be taken from a sizable borrow pit.

- The left abutment might contain pervious zones needing treatment to reduce seepage losses and improve abutment stability.

- Deep cuts are required for the spillway approach channel, for the stilling basin, and for embankment foundation preparation.

- The seismic activity of the region would require flat slopes for the embankment.

- A wide chimney drain and large downstream drainage blanket would be required because of the potential for ground shaking from earthquakes.

- The material available for core construction is susceptible to cracking and would be sensitive to moisture content during construction.

- For embankment safety during earthquakes, the dam must be carefully constructed under strict quality control for materials and compaction.

- Filter and riprap bedding materials must be processed from selected onsite materials.

- Riprap and granular roadway surfacing must be imported from offsite.

- The construction season is relatively short, and maximum river flows occur during the middle of the construction season.

- The river diversion would require close attention by the contractor because of the narrow valley and the possibility of large flood flows during construction.

- Dewatering would be necessary in the lower parts of the foundation excavation and core trench. Large flows may have to be handled if pervious zones are present.

CONSTRUCTION SEQUENCE

The construction sequence would be the contractor's responsibility. This sequence would depend on his experience, the type of equipment he has available for the project, the time required for delivery of items required for construction, and other factors. The sequence adopted should provide good construction conditions and minimize the possibility that the partially completed dam might be overtopped during construction. For project analysis, we have used the following generalized construction sequence:

1. Construct a cofferdam around the low-level outlet and stilling basin construction areas, and divert the Eagle River to the extreme left side of the valley.

2. Construct the low-level outlet and the spillway stilling basin.

3. Construct cofferdams to protect the embankment and spillway work areas, and divert the river through the low-level outlet.

4. Prepare the foundation for the dam and the spillway.

5. Construct the main dam embankment, using material from the spillway excavation to the maximum extent possible in the embankment. Construct the spillway at the same time.

6. Install major equipment and begin reservoir operation.

CONSTRUCTION SCHEDULE

We estimate that construction of the dam would take approximately 2 years under the following conditions:

- o No unusual conditions encountered that are not now apparent or that differ from assumptions made.

- o Work is properly organized by the contractor.

- o The contract is awarded early enough before the construction season begins to allow time for the contractor to organize the work and order items with long lead time (such as the radial gates). This will also allow the maximum number of days of good weather for construction, and allow the contractor to take advantage of seasonal streamflow variations in developing his work plan.

- o Access routes to all work areas are available to the contractor.

- o All necessary property not now owned by the Municipality of Anchorage is acquired, or easements are acquired, before construction begins.

- o All permits required are issued in a timely manner by the responsible agencies.

Other than unusual circumstances such as acts of God, strikes, unusual weather conditions, or other unforeseen circumstances, the completion time is dependent on contractor operations. A competent, experienced contractor with adequate plans should be able to complete the project in 2 years without charging a premium price for a "rush" job.

DIVERSION AND CARE OF WATER

The handling of natural streamflow during construction of a dam is usually a key element of the work. The contractor would have control of the project site during construction, and would be required to provide for diversion of the river flow during construction. Because the diversion cannot be economically constructed to handle all flows, there is a risk that the construction site might be flooded, with resulting damages and delays. This is a risk for the contractor and, thus, for the owner.

We anticipate that the contractor would divert water away from the construction area by first diverting it to the left side of the valley while the outlet conduits and stilling basin are being constructed. Once these components are finished, they can be used in conjunction with an upstream cofferdam to pass streamflow until the dam is completed. The contractor might elect to use other methods to divert the river.

We anticipate that the proposed core and shell materials would be sensitive to moisture content during compaction. This would require close moisture control to achieve satisfactory compaction. During construction the contractor may find that the natural moisture content of materials in the borrow area is too high or too low for satisfactory compaction. This would then require either removal or addition of water.

During excavation of the core trench and for the foundations of the dam and the structures, springs may develop that may require drainage or pumping.

COST ESTIMATE

The estimated cost for the construction of Eagle River Dam is based on estimated quantities of materials and construction services required for execution of the conceptual design presented in this report. The cost estimate is presented in Table 9-1. These costs reflect April 1981 cost levels, and are based on our past experience and records. They would require escalation beyond April 1981 to account for inflation. The estimate given in Table 9-1 includes only costs for construction and engineering. It does not include items such as legal fees, easement acquisition, property acquisition, and financing costs. It also does not include the costs of fish facilities. These additional costs also will depend on the assumptions listed in the Construction Schedule section of this chapter.

We believe that these are reasonable conceptual design cost estimates, but they do not constitute a quotation or guaranteed maximum because of the many uncertainties and unknowns that are beyond our control, such as the following:

- The final design of the dam might differ from the conceptual design presented in this report.

- Fish facility requirements have not been established. Consequently, the cost of such facilities has not been included.

Table 9-1
CONSTRUCTION COST ESTIMATE

	Costs by Category ($)					Facility
	Mobilization	Sitework	Concrete	Equipment	Electrical	Totals
Site Access	5,000	105,000	--	--	--	110,000
Dewatering	27,000	545,000	--	--	--	572,000
Dam Embankment	103,000	2,054,000	--	--	--	2,157,000
Reservoir Preparation	293,000	5,865,000	--	--	--	6,158,000
Spillway	387,000	448,000	5,696,000	1,603,000	250,000	8,384,000
Outlet Conduits	74,000	--	1,484,000	--	--	1,558,000
Control Tower	60,000	--	1,035,000	174,000	--	1,269,000
Totals	949,000	9,017,000	8,215,000	1,777,000	250,000	20,208,000
Engineering at 15 percent						3,032,000
Total						23,240,000

Notes:
1. All items include 25 percent contingency.
2. Costs do not include legal fees, easement acquisition, property. acquisition, financing costs, and fish facilities.
3. All costs in April 1981 dollars.

- The effects of frazil ice on design, sedimentation, and water quality must be studied further. Changes in the final design might be required as a result of such studies. These changes might affect the cost of the project.

- There are uncertainties in any cost estimate

- Uncertainties in the competitive bidding process can have a strong effect on bid prices. Some factors that might affect the bid prices include the amount of other work available to contractors during the bidding period, the amount of backlog work contractors have at the time of the bidding, the number of contractors bidding on the project, the contractors' estimates of the difficulty of constructing the project, and the contractors' financing requirements for the project in periods of inflation.

As discussed previously, the construction of a dam involves a greater degree of uncertainty than most engineering projects. When changes during construction are necessary, they need to be reviewed, analyzed, and approved rapidly to allow construction to proceed. This type of change generally requires "change orders" to the construction contract. These change orders generally cause an increase in the project cost over the basic bid. The contingency amount provided in the cost estimate is intended to provide a reserve fund to meet at least part of the costs of changes, if required, and also as a contingency for other uncertainties inherent in the estimating and bidding processes, as described above.

PERMITS

A number of permits or similar authorizations from governmental agencies would be required for the construction of the Eagle River dam. The following is a list of agencies and permits that would or may be required:

U.S. Army Corps of Engineers

- Permit for Discharge of Dredged or Fill Material into Waters of the U.S.

Alaska Department of Environmental Conservation

- Water quality certification

- Surface oiling permit

- Solid waste disposal permit

Alaska Department of Fish and Game

- o Habitat permit
- o Anadromous fish protection permit
- o Determination of need for fishways resulting from obstruction of fish passage

Alaska Department of Natural Resources

- o Water rights permit
- o Compliance with dam safety regulations
- o If state lands are affected:
  - Miscellaneous land use permit
  - Right-of-way or easement permit
- o If state park lands are affected:
  - State park noncompatible use permit
  - Disturbance of natural material permit

Municipality of Anchorage

- o Miscellaneous permits that may include building permits and right-of-way permits

In addition, the U.S. Environmental Protection Agency may require an Environmental Assessment or an Environmental Impact Statement.

The Office of the Governor, Division of Policy Development and Planning (Alaska Coastal Management Program), would make a "consistency determination." This consists of a review of applications for Federal licenses and permits, circulation of the applications and information to the appropriate state agencies for review and comment, and a determination of consistency with state-level authority and regulations. The results of this consistency determination are submitted to the appropriate Federal agency.

If AWSU decides to construct the Eagle River dam, the permit application process should be one of the first tasks in the final design process.

CONSTRUCTION REVIEW

A full-time construction reviewer should be present at the damsite during construction to continually evaluate field conditions encountered during construction and compare them with the design assumptions. This reviewer should be familiar with the design, with the geotechnical assumptions made during design, and with dam design and construction in general. The presence of a construction reviewer helps to identify differing conditions that are encountered during construction so that they may be properly addressed by the design engineers. He monitors the construction to evaluate its conformance to the plans, specifications, and any change orders that may be necessary.

Chapter 10
OPERATION AND MAINTENANCE

Ownership of a dam creates a continuing responsibility for the operation and maintenance of the dam. This chapter discusses some general considerations related to operation and maintenance of the proposed dam.

The final design engineering tasks should include development of a detailed operation and maintenance plan.

OPERATION

Normal Operation

The normal hydraulic operation plan for the dam is described in Chapter 3. It is anticipated that all of the reservoir control systems would be remotely operated from the proposed Eagle River water treatment plant. In addition, local override of the remote controls would be possible. A daily visit to the damsite by AWSU staff should be made to observe the condition of all facilities.

Instrumentation should be provided to monitor reservoir levels, the groundwater elevation in the dam and the abutments, and the movement and settlement of the dam. Piezometers or observation wells would be used to measure the water pressure in various portions of the dam and the abutments, and survey monuments would be used to measure any movement or settlement of the fill. These measurements would provide data for permanent records that can be used to determine if the design assumptions are being met and to evaluate the long-term performance of the dam. These data would also provide background information for safety evaluations or repairs that might be needed at a future date. In addition, the monitoring of reservoir levels would be used in the daily operation of the project.

The conceptual design of the spillway allows for controlled opening of the spillway gates at rates less than or equal to a predetermined rate so that downstream river levels would not rise rapidly. For this study, the rate of downstream river rise was limited to 2 feet per hour or less. This would allow sufficient time for persons downstream to leave the river area in the event of increasing discharges.

Emergency Operation

As mentioned previously, local override of all control systems would be possible at the damsite. In addition, backup operating power must be provided at the damsite by standby generators.

The three radial spillway gates can be fully opened, if needed, to an elevation above the surface of the water that is discharging through the spillway during an extreme flood. However, as mentioned above, the spillway gates would normally be opened at a rate that would limit the maximum rise in downstream river level to 2 feet or less per hour. The spillway would be designed to operate under all flood conditions up to the routed peak discharge of the PMF. However, it is expected that some damage might occur to the stilling basin as a result of discharges exceeding one-half the PMF peak discharge. This is because it is economically impractical to provide a stilling basin that would suffer no damage during such an extremely unlikely event. It is anticipated that repairs would be required to the stilling basin and possibly to the downstream toe of the dam embankment and fish facilities in the event that a flow of this magnitude occurs.

It is important to operate the reservoir at all times to ensure that the normal reservoir level does not exceed elevation 338 feet. If the normal reservoir level is not held near or below this elevation, there would be diminished storage available in the reservoir to store the excess inflow which would occur during the routing of the PMF. The reservoir level should be permitted to rise above normal only if all three radial gates are already fully opened and clear of the water surface, or to limit downstream water level increases to 2 feet per hour as discussed above. When the reservoir is full, there would be very small increases in water level above elevation 338 as needed for automatic reservoir sensors to detect a rising reservoir level. During floods that occur when the reservoir is full, the spillway gates would be opened to match outflow with reservoir inflow.

EMERGENCY WARNING PLAN

A flood warning plan would be required for residents downstream of the dam and for closure of the Glenn Highway and Alaska Railroad bridges downstream of the dam. It would be used during severe flooding or in the unlikely event of potential dam failure. A dam break analysis and routing of the resultant flood wave would be required to determine the extent of potential flooding downstream of the dam.

The emergency plan should clearly indicate the discharge and/or reservoir elevation at which steps would be taken to evacuate downstream residents and close these facilities. The plan should also identify other conditions that would require its use. Responsible government authorities or other agents who would agree to accept responsibility to carry out the flood warning plan must be identified. The emergency plan should be written, available in a number of places, and be conspicuously posted at the dam and at the water treatment plant control center. It should identify a series of backup personnel in the event the prime contact personnel cannot be reached.

MAINTENANCE

Both preventive and corrective maintenance would be required for the dam and control structures. Preventive maintenance would be necessary to keep the dam and control structures in satisfactory operating condition. Corrective maintenance may be required to repair the dam and control structures if they become damaged.

A maintenance plan should be developed to provide routine guidelines for the preventive maintenance of the structure. Some of the elements that would be part of a preventive maintenance plan are:

- Periodic lubrication of gate hoists
- Periodic inspection and replacement of hoist cables
- Periodic maintenance of control systems, instrumentation, and actuators
- Maintenance and repair of the roadways
- Periodic mowing or brush control
- Correction of minor deficiencies such as cracks or concrete spalling as they develop and before they progress to the point of causing major damage
- Periodic painting of exposed metalwork
- Periodic collection of debris from log booms and trashracks
- Daily monitoring of ice conditions to allow early removal of unusual ice buildup that may hamper project operation

Examples of the types of corrective maintenance that may be required include:

- The abutments should be inspected regularly for the development of springs and presence of seepage water. If this occurs, means to control the seepage would be required. Control can often be accomplished by means of an upstream blanket of low permeability soil such as silt or clay over the area of infiltration in the reservoir. In the event of excessive leakage, drainage trenches, blankets, pipes, or vertical or horizontal drainage wells might be required.
- Corrosion of the gates, valves, and piping would gradually occur. These items should be inspected periodically and repaired as needed.

- Erosion of concrete in the stilling basin and around the radial and roller gates would occur because of the high velocity of the water. Structures subjected to high velocity water should be inspected periodically and repaired as needed.

- During large floods, erosion is expected in the stream channel near the spillway stilling basin. It may be necessary to place additional fill in the vicinity of the stilling basin if it is undermined or otherwise threatened by such erosion.

- Settlement of the dam may cause some visible cracks to develop on the surface. These cracks should be observed, measured, and repaired. This cracking and differential settlement is expected to occur within the first few years of reservoir operation.

INSPECTION

An ongoing inspection program is essential to the integrity of a major dam such as the proposed Eagle River dam. Such an inspection program should include both an informal and a formal program of inspection.

The informal program is often the most important. The informal program requires that the normal operating personnel be conscious of the day-to-day condition of the dam and of specific features that have been identified as potential hazards. In this manner, changes in site conditions can be noted and evaluated promptly.

The formal inspection program should consist of a regularly scheduled systematic inspection of all features of the structure. This inspection should involve formal documentation, such as completion of a checklist developed specifically for Eagle River dam. In general, photographs of the dam should be taken from the same vantage points at each inspection. This type of inspection provides a frame of reference against which to evaluate future changes in the dam's condition. The formal inspection should be performed at least annually. In addition, a formal inspection should be performed during or after every instance of unusually high water conditions.

The establishment of a program of formal inspections with photographs and records of the dam's condition assembled on a regular basis and kept indefinitely can aid significantly in future evaluation of the dam's safety.

It is also advisable to have a formal inspection conducted at 5-year intervals by an outside agency or consultant having no interest in the day-to-day operation of the dam. Such an inspection provides an independent review of the dam's safety and condition.

EFFECTS OF FUTURE CHANGES

Under the criteria and assumptions given in this report, the dam and reservoir are designed for safe and efficient operation. Changes from these criteria or assumptions may produce undesirable or unsafe conditions in the dam and reservoir or downstream. Future developments in science and engineering might create a need to reevaluate the design criteria used. Changes in land use either upstream or downstream of the dam might change the design assumptions, and might create pressure to change reservoir operation procedures. Any changes in reservoir operation method, reservoir operation elevations, or adjacent construction should be carefully evaluated by qualified professionals to determine the possible effects of such changes on the dam and reservoir.

Successful evaluation of the effects of changed conditions or future developments can only be adequately performed by qualified professionals experienced in the design and operation of earthfill dams.

Chapter 11
CONCLUSIONS AND RECOMMENDATIONS

As a result of the Eagle River preliminary damsite investigation the following conclusions and recommendations are presented.

CONCLUSIONS

A dam can be constructed on the Eagle River that can be safely operated to provide municipal water supply and to safely withstand the maximum credible earthquake and the probable maximum flood. The dam can be constructed to provide a water supply withdrawal rate of up to 108 cfs (70 mgd) if there are no major deviations from the following assumptions used in the investigation:

- 31 cfs is adequate for minimum downstream releases
- Mitigation for fisheries can be achieved to the satisfaction of controlling agencies
- Other environmental elements will not block construction of the dam or withdrawal of water from the river
- Sediment deposition in the reservoir will not occur at a rate that will make the dam infeasible
- All permits and licenses can be obtained from the appropriate agencies

A dam that provides for a water supply withdrawal rate of 108 cfs is estimated to cost $23,240,000 in April 1981 dollars. This amount is for construction and engineering only, and does not include:

- Land acquisition
- Financing
- Escalation to adjust for higher prices at the future construction date
- Fish facilities

RECOMMENDATIONS

Many uncertainties encountered during the preliminary damsite investigation need to be resolved by additional studies before any dam on the Eagle River can be designed. If AWSU decides to pursue the Eagle River dam, we recommend the following studies or actions be completed prior to or during design:

- Study the type, number, migration, and distribution of fish in the river to evaluate impacts of a dam and to provide satisfactory mitigation of the impacts. This study should lead to a decision on required minimum releases from the dam, by month if necessary, before design of the dam, because the size of the dam and reservoir is dependent on minimum release requirements.

- Study the potential effect of the old Eagle River dump on reservoir water quality and evaluate any modifications required to develop the Eagle River as a water source.

- Study the variability of the water supply demand throughout the year, because a variable rate can affect dam size.

- Install at least one precipitation station in the upper reaches of the basin to provide hourly precipitation values, and modify other stations in and around the basin to provide hourly precipitation values.

- Recompute the PMF during design.

- Study the winter regime of the Eagle River.

- Perform a more detailed seismicity study for the site to determine the characteristics of the maximum credible earthquake and the dynamic response of the site.

- Perform additional subsurface exploration surrounding the damsite to evaluate in detail the questions of seepage around the abutments and to aid in construction management of stream underflow water and in design of the embankment underdrain system.

- Perform additional borrow exploration and testing for select materials such as core and filter materials.

- Study the effect on water treatability of not stripping the surface vegetation and topsoil from the reservoir area.

- Conduct a sediment sampling study to provide data for reservoir sedimentation estimates.

Chapter 12
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Exhibit A
Streamflows

EAGLE RIVER
MONTHLY STREAMFLOW SIMULATION
TRIAL NO.2 HISTORICAL ONLY

IYRA	IMNTH	IANAL	MXRCS	NYRG	NYMXG	NPASS	IPCHQ	IPCHS	NSTA	NCOMB	NTNDM	NCSTY	IGNRL	NPROJ	IYRPJ	MTHPJ	LYRPJ
1966	10	1	14	196	14	1	0	0	0	0	0	0	0	0	0	0	0

MAXIMUM VOLUMES OF RECORDED FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	707	190	137	118	97	85	117	366	1507	2120	2424	1593	2424	7980	32199	516

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	155	86	54	39	26	36	36	82	662	1290	874	430	26	322	21412	

FREQUENCY STATISTICS

STA	ITEM	10	11	12	1	2	3	4	5	6	7	8	9
771	MEAN	2.480	2.098	1.941	1.812	1.728	1.720	1.843	2.361	2.973	3.228	3.201	2.912
	STD DEV	0.177	0.108	0.122	0.142	0.158	0.115	0.114	0.174	0.114	0.069	0.120	0.178
	SKEW	0.374	0.265	0.350	0.217	-0.109	0.616	-0.745	-1.215	0.438	-0.127	-0.266	-0.233
	INCRMT	3.24	1.28	0.90	0.67	0.56	0.54	0.71	2.43	9.62	16.95	16.29	8.72
	YEARS	14	14	14	14	14	14	14	14	14	14	14	14

RAW CORRELATION COEFFICIENTS FOR MONTH 10

STA 771
WITH CURRENT MONTH
771 1.000
WITH PRECEDING MONTH AT ABOVE STATION
771 0.244

RAW CORRELATION COEFFICIENTS FOR MONTH 11

STA 771
WITH CURRENT MONTH
771 1.000
WITH PRECEDING MONTH AT ABOVE STATION
771 0.667

RAW CORRELATION COEFFICIENTS FOR MONTH 12

STA 771
WITH CURRENT MONTH
771 1.000
WITH PRECEDING MONTH AT ABOVE STATION
771 0.842

RAW CORRELATION COEFFICIENTS FOR MONTH 1

STA 771
WITH CURRENT MONTH
771 1.000

```
                       WITH PRECEDING MONTH AT ABOVE STATION
  771  0.853

RAW CORRELATION COEFFICIENTS FOR MONTH  2

  STA    771
                       WITH CURRENT MONTH
  771  1.000
                       WITH PRECEDING MONTH AT ABOVE STATION
  771  0.912

RAW CORRELATION COEFFICIENTS FOR MONTH  3

  STA    771
                       WITH CURRENT MONTH
  771  1.000
                       WITH PRECEDING MONTH AT ABOVE STATION
  771  0.866

RAW CORRELATION COEFFICIENTS FOR MONTH  4

  STA    771
                       WITH CURRENT MONTH
  771  1.000
                       WITH PRECEDING MONTH AT ABOVE STATION
  771  0.449

RAW CORRELATION COEFFICIENTS FOR MONTH  5

  STA    771
                       WITH CURRENT MONTH
  771  1.000
                       WITH PRECEDING MONTH AT ABOVE STATION
  771  0.701

RAW CORRELATION COEFFICIENTS FOR MONTH  6

  STA    771
                       WITH CURRENT MONTH
  771  1.000
                       WITH PRECEDING MONTH AT ABOVE STATION
  771  0.520

RAW CORRELATION COEFFICIENTS FOR MONTH  7

  STA    771
                       WITH CURRENT MONTH
  771  1.000
                       WITH PRECEDING MONTH AT ABOVE STATION
  771  0.673

RAW CORRELATION COEFFICIENTS FOR MONTH  8
```

STA 771

WITH CURRENT MONTH

771 1.000

WITH PRECEDING MONTH AT ABOVE STATION

771 0.791

RAW CORRELATION COEFFICIENTS FOR MONTH 9

STA 771

WITH CURRENT MONTH

771 1.000

WITH PRECEDING MONTH AT ABOVE STATION

771 0.682

1

RECORDED AND RECONSTITUTED FLOWS

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	1966	425	162	83	51	50	50	51	235	1150	1858	1958	1177	7250
771	1967	342	103	77	67	51	53	75	255	1507	2116	2221	1593	8460
771	1968	288	133	82	72	64	61	63	356	961	1775	1450	476	5781
771	1969	155	107	70	39	32	42	79	322	1252	1564	874	457	4993
771	1970	707	174	119	99	90	85	78	218	739	1303	1241	588	5441
771	1971	181	103	75	57	52	40	36	82	725	1772	2002	552	5677
771	1972	191	100	90	65	39	36	59	145	689	1747	1589	970	5720
771	1973	418	143	123	73	48	44	71	160	662	1290	1227	430	4689
771	1974	267	86	54	39	26	40	77	272	921	1472	1489	1141	5884
771	1975	231	123	81	48	45	45	77	313	746	1652	1307	756	5424
771	1976	237	91	65	56	53	46	64	177	889	1615	1386	900	5579
771	1977	307	190	130	94	79	62	78	249	1333	2120	2424	1100	8166
771	1978	460	158	137	118	97	79	72	246	816	1447	1528	971	6129
771	1979	324	116	72	65	61	69	117	366	1082	2001	2103	1098	7474

1

GENERATED FLOWS FOR PERIOD 1

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	1	251	114	78	40	23	36	23	35	572	1146	1200	344	3862
771	2	179	100	62	36	21	29	54	189	673	1367	1246	1054	5010
771	3	289	114	80	52	41	42	79	323	769	1965	1793	573	6120
771	4	187	88	65	43	39	47	68	204	1055	1774	1458	549	5577
771	5	339	130	81	68	60	48	68	207	840	1911	1690	870	6312
771	6	589	129	100	81	72	78	83	352	901	1631	1469	362	5847
771	7	141	93	72	51	48	41	55	205	713	1643	1786	1503	6351
771	8	264	115	83	48	29	39	51	252	756	1578	1498	734	5447
771	9	142	87	80	58	43	45	53	131	776	1353	919	614	4301
771	10	229	97	64	42	41	40	52	129	899	1581	1144	677	4995
771	11	287	92	59	42	33	40	70	170	1070	1730	1721	981	6295
771	12	317	203	108	74	66	51	46	140	973	1849	1658	838	6323
771	13	416	138	81	73	63	61	82	356	834	1560	1291	661	5616
771	14	360	101	60	36	32	31	43	322	1149	1673	1451	537	5795

1

MAXIMUM VOLUMES FOR PERIOD 1 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	589	203	108	81	72	78	83	356	1149	1965	1793	1503	1965	6115	29775	463

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	141	87	59	36	21	29	23	35	572	1146	919	344	21	234	21112	

1

GENERATED FLOWS FOR PERIOD 2

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	15	207	111	67	66	47	39	73	278	1127	1884	1867	1089	6855
771	16	265	107	70	54	38	46	70	209	747	1444	1153	481	4684
771	17	288	121	93	72	70	55	91	333	950	1545	1316	427	5361
771	18	372	142	104	72	48	47	50	123	862	1596	1612	639	5667
771	19	289	109	49	44	34	43	77	182	791	1449	1134	720	4921
771	20	245	112	117	101	107	75	60	171	676	1574	1414	885	5537
771	21	303	111	84	81	59	53	82	289	1371	2140	1898	982	7453
771	22	316	122	83	61	43	52	64	286	1254	1836	1868	1056	7041
771	23	301	102	76	43	39	38	66	132	794	1421	1386	558	4956
771	24	337	145	73	66	57	68	77	325	987	1569	1252	617	5573
771	25	214	93	62	42	24	39	38	70	784	1728	1409	760	5263
771	26	727	171	100	63	57	51	71	336	1093	2041	2042	1489	8241
771	27	454	198	139	134	111	94	70	219	1114	1542	1358	659	6092
771	28	476	121	88	85	58	49	101	346	1276	2112	2642	1619	8973

1

MAXIMUM VOLUMES FOR PERIOD 2 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	727	198	139	134	111	94	101	346	1371	2140	2642	1619	2642	8095	33393	516

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	207	93	49	42	24	38	38	70	676	1421	1134	427	24	274	21183	

1

GENERATED FLOWS FOR PERIOD 3

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	29	519	171	145	92	52	50	67	131	822	1742	1794	789	6374
771	30	245	132	114	89	70	62	99	295	1159	1907	1646	667	6485
771	31	181	86	64	40	28	35	49	158	678	1690	1240	768	5017
771	32	202	105	62	51	36	40	91	314	852	1463	1492	720	5428
771	33	273	106	82	58	56	51	77	268	920	1811	1330	658	5690
771	34	238	141	92	69	49	49	56	159	1007	1829	1821	712	6222
771	35	325	98	85	64	73	70	69	248	1070	1715	1342	541	5700
771	36	357	140	82	61	56	48	77	319	925	1401	1525	1089	6080
771	37	323	131	111	117	107	122	109	421	1487	2543	2274	838	8583
771	38	403	148	92	73	58	67	84	383	1179	1817	2222	1196	7722
771	39	197	101	60	44	41	38	51	241	980	1808	1298	653	5512
771	40	566	184	127	69	72	57	57	84	621	1625	1557	949	5968
771	41	260	130	92	75	57	54	57	99	1151	1743	1576	857	6151
771	42	482	182	157	111	96	104	94	233	599	1305	1172	454	4989

1

MAXIMUM VOLUMES FOR PERIOD 3 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	566	184	157	117	107	122	109	421	1487	2543	2274	1196	2543	7966	33812	511

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	181	86	60	40	28	35	49	84	599	1305	1172	454	28	301	22958	

1

GENERATED FLOWS FOR PERIOD 4

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	43	242	153	93	68	59	54	56	234	782	1434	1214	506	4895
771	44	163	79	49	34	20	28	24	57	605	1237	935	441	3672
771	45	278	123	96	82	55	52	70	313	1020	2037	1764	1366	7256

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	46	257	150	88	88	69	53	78	343	1271	1742	1465	1147	6751
771	47	238	133	79	69	52	50	55	154	713	1438	1757	820	5558
771	48	460	136	97	77	62	49	79	305	1145	1531	1085	679	5705
771	49	296	116	96	76	75	64	102	358	1340	2056	1524	669	6772
771	50	512	194	99	75	61	60	89	253	1347	1873	2000	723	7286
771	51	135	94	58	38	33	39	63	126	730	1595	1363	612	4886
771	52	246	89	55	36	26	34	37	83	579	1506	837	569	4097
771	53	190	112	77	51	46	46	101	314	835	1596	1380	899	5647
771	54	454	138	88	60	44	44	62	275	681	1373	1372	838	5429
771	55	212	113	71	75	99	95	68	324	1361	2149	2353	1707	8627
771	56	236	127	106	70	52	57	71	172	1253	2064	2950	1313	8471

1

MAXIMUM VOLUMES FOR PERIOD 4 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	512	194	106	88	99	95	102	358	1361	2149	2950	1707	2950	8130	31797	506

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	135	79	49	34	20	28	24	57	579	1237	837	441	20	211	20659	

1

GENERATED FLOWS FOR PERIOD 5

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	57	239	152	103	70	66	70	83	348	998	1841	1236	683	5889
771	58	292	140	128	140	102	73	106	392	1073	1696	1623	971	6736
771	59	308	156	142	119	104	98	85	305	906	1670	2005	1137	7035
771	60	410	151	97	59	40	52	70	146	889	1806	1825	877	6422
771	61	221	112	88	66	64	57	74	167	888	1644	1433	798	5632
771	62	337	172	142	88	64	54	67	184	993	1621	1288	333	5343
771	63	242	113	117	87	74	69	107	415	1014	1562	1490	772	6062
771	64	268	116	97	81	65	56	87	330	783	1573	1101	597	5174
771	65	216	93	55	43	41	43	84	179	913	1546	1201	582	4996
771	66	600	226	210	188	126	137	96	252	829	1440	1541	718	6363
771	67	288	134	101	69	52	51	60	156	595	1351	1582	820	5259
771	68	289	127	74	55	37	38	75	245	966	1802	2398	1273	7379
771	69	606	162	90	83	53	48	65	159	1041	2051	1813	1021	7192
771	70	505	134	90	70	50	59	79	255	1057	1781	1869	1285	7234

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MAXIMUM VOLUMES FOR PERIOD 5 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	606	226	210	188	126	137	107	415	1073	2051	2398	1285	2398	7291	31938	516

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	216	93	55	43	37	38	60	146	595	1351	1101	333	37	359	22197	

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GENERATED FLOWS FOR PERIOD 6

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	71	276	115	70	45	38	44	54	181	781	1224	730	860	4418
771	72	674	201	148	84	78	64	58	255	896	1500	1177	713	5848
771	73	228	119	101	61	55	50	62	235	864	1445	1289	596	5105
771	74	474	113	73	54	45	47	42	68	726	1555	1114	527	4838
771	75	159	87	53	40	33	43	47	278	959	1747	2084	1113	6643
771	76	388	113	78	69	52	44	66	194	1228	1657	1837	971	6697
771	77	335	154	121	95	106	65	101	351	956	1696	1465	609	6054
771	78	272	108	74	58	37	42	91	232	787	1608	1278	502	5089
771	79	315	145	86	87	68	71	66	185	1055	1937	1957	1745	[illegible]

771	80	262	80	66	41	29	39	78	343	1075	1802	1397	921	6133
771	81	737	230	197	159	119	78	104	414	1999	2261	1953	1071	9322
771	82	163	91	66	55	47	64	86	265	1021	1733	1971	780	6342
771	83	321	128	98	62	52	59	67	201	855	1377	1616	1107	5943
771	84	451	135	79	46	37	49	74	257	782	1613	1096	843	5462

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MAXIMUM VOLUMES FOR PERIOD 6 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	737	230	197	159	119	78	104	414	1999	2261	2084	1765	2261	7861	35085	510

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	159	80	53	40	29	39	42	68	726	1224	730	502	29	303	20394	

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GENERATED FLOWS FOR PERIOD 7

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	85	236	92	77	56	49	44	89	287	1179	2018	2265	1447	7839
771	86	288	96	84	67	49	42	65	325	1322	2117	2558	1682	8695
771	87	277	112	87	81	61	56	55	187	957	1953	1807	709	6342
771	88	254	127	94	67	51	51	94	338	808	1536	1429	1145	5994
771	89	436	199	151	102	62	66	122	420	1040	1841	1664	822	6925
771	90	216	90	56	35	29	39	43	155	893	1477	1438	784	5255
771	91	269	120	73	52	38	46	60	189	1072	1625	1726	909	6179
771	92	185	120	86	74	74	64	93	246	873	1322	949	501	4587
771	93	229	128	89	69	53	46	47	296	775	1910	1611	826	6079
771	94	591	193	128	119	105	88	96	262	1158	2042	1964	865	7611
771	95	203	115	71	46	43	53	62	241	924	1924	1867	988	6537
771	96	316	154	96	66	67	65	69	185	1012	1748	1315	912	6005
771	97	346	96	64	48	36	48	82	370	1512	1997	1561	816	6976
771	98	430	121	74	46	30	35	62	206	944	1423	1252	996	5619

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MAXIMUM VOLUMES FOR PERIOD 7 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	591	199	151	119	105	88	122	420	1512	2117	2558	1682	2558	8280	35367	540

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	185	90	56	35	29	35	43	155	775	1322	949	501	29	291	23170	

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GENERATED FLOWS FOR PERIOD 8

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	99	294	165	104	58	46	45	66	289	1589	1783	1977	968	7384
771	100	411	118	108	84	71	61	80	294	992	1904	2062	1019	7204
771	101	181	92	62	51	43	38	69	106	805	1529	1306	569	4851
771	102	359	200	137	103	97	101	83	340	833	1679	1322	1045	6299
771	103	757	150	95	60	57	41	81	209	896	1594	1542	479	5961
771	104	253	100	56	40	41	42	82	315	660	1602	1588	739	5518
771	105	165	123	89	127	110	84	78	334	745	1197	1145	630	4827
771	106	430	148	114	75	46	53	61	218	822	1668	1521	842	5998
771	107	215	112	59	48	45	41	65	253	1209	1961	1983	674	6665
771	108	550	185	123	80	79	57	62	329	1212	2072	1498	781	7028
771	109	409	144	100	72	62	59	60	288	783	1691	1519	699	5889
771	110	241	135	116	67	61	56	81	221	1000	1969	2015	821	6783
771	111	440	245	117	97	73	63	68	339	764	1299	1080	900	5485
771	112	293	87	51	36	32	33	47	214	1575	2121	2743	1885	9117

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MAXIMUM VOLUMES FOR PERIOD 8 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	757	205	137	127	110	101	98	340	1675	2121	2743	1885	2743	8675	33227	530

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	165	87	51	36	32	33	47	106	660	1197	1080	479	32	286	22130	

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GENERATED FLOWS FOR PERIOD 9

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	113	230	126	75	58	45	59	54	221	822	1516	1286	829	5321
771	114	335	109	75	85	78	75	97	287	1264	1972	2347	2096	8820
771	115	437	123	80	49	34	45	99	376	1016	1737	1847	680	6523
771	116	276	120	92	63	53	51	42	139	1002	1637	1355	523	5353
771	117	280	130	84	53	52	51	63	271	827	1691	1699	622	5823
771	118	183	114	106	92	79	59	83	290	754	1338	843	463	4404
771	119	337	106	73	44	37	35	46	63	653	1796	2344	1539	7073
771	120	327	160	104	68	63	62	60	273	806	1856	1949	1487	7295
771	121	538	110	64	46	36	40	66	314	1166	1714	1970	770	6834
771	122	225	124	118	97	108	99	61	192	713	1597	2096	1176	6606
771	123	322	112	75	42	29	35	57	151	895	1927	2128	1130	6903
771	124	396	109	79	61	62	57	61	145	693	1375	1409	828	5275
771	125	483	161	112	94	75	62	92	252	772	1565	1945	909	6522
771	126	858	127	85	69	63	65	72	180	1044	1730	1365	467	6125

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MAXIMUM VOLUMES FOR PERIOD 9 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	858	161	118	97	108	99	99	376	1264	1972	2347	2096	2347	8403	34479	529

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	183	106	64	42	29	35	42	63	653	1338	843	463	29	298	22283	

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GENERATED FLOWS FOR PERIOD 10

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	127	236	106	66	53	59	56	45	81	556	1067	922	443	3690
771	128	285	141	104	82	58	55	99	284	1152	2070	2462	782	7574
771	129	633	159	123	69	49	41	70	235	1056	1672	1731	1066	6904
771	130	200	102	72	80	68	64	74	312	1201	2026	1718	724	6641
771	131	190	112	95	79	65	57	45	176	839	1316	1002	856	4832
771	132	291	143	114	68	64	40	87	257	1146	2204	1906	1480	7800
771	133	288	119	98	76	52	45	61	214	885	1729	1706	974	6247
771	134	586	137	97	72	72	62	74	211	1416	1710	1577	908	6922
771	135	289	86	72	43	34	45	48	165	767	1414	1457	354	4774
771	136	235	158	89	86	84	73	74	267	831	1499	1267	479	5142
771	137	237	121	87	62	50	56	80	254	879	1607	1247	603	5283
771	138	155	105	76	48	49	55	62	235	1223	2087	2040	958	7093
771	139	270	113	75	68	52	45	77	356	825	2039	2040	1015	6975
771	140	205	92	55	45	44	58	94	349	952	2167	2347	1096	7504

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MAXIMUM VOLUMES FOR PERIOD 10 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	633	159	123	86	84	73	99	356	1416	2204	2462	1480	2462	7384	33215	520

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	155	86	55	43	34	40	45	81	556	1067	922	354	34	329	22085	

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GENERATED FLOWS FOR PERIOD 11

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	141	319	124	61	54	42	40	58	171	643	1150	1070	857	4589
771	142	311	146	107	89	61	68	51	149	589	1305	797	349	4022
771	143	223	155	105	112	114	104	80	250	877	1885	1350	529	5784
771	144	210	130	89	69	49	52	89	349	1226	2084	1621	643	6611
771	145	239	80	59	44	41	50	59	205	770	1405	1392	531	4875
771	146	242	96	75	68	59	57	93	279	1123	1628	1448	670	5838
771	147	432	140	81	65	48	48	79	159	829	1681	1452	818	5832
771	148	208	124	74	69	67	73	104	349	1001	1309	1025	416	4819
771	149	274	186	122	72	53	75	87	229	1092	1774	1872	1033	6869
771	150	296	144	128	110	89	73	82	294	1225	1571	1601	511	6124
771	151	264	99	59	40	32	39	80	270	1059	1412	934	405	4693
771	152	444	161	110	80	59	42	38	145	1114	2161	1711	1180	7245
771	153	327	115	96	84	80	74	72	275	844	1739	1326	464	5496
771	154	220	126	98	62	58	57	66	125	839	1621	1276	1116	5664

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MAXIMUM VOLUMES FOR PERIOD 11 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	444	186	128	112	114	104	104	349	1226	2161	1872	1180	2161	6638	29780	467

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	208	80	59	40	32	39	38	125	589	1150	797	349	32	334	21257	

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GENERATED FLOWS FOR PERIOD 12

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	155	189	100	72	60	36	33	60	129	955	1746	1734	530	5644
771	156	671	192	105	67	48	45	77	260	980	1791	1923	1287	7446
771	157	208	105	84	83	79	88	65	251	1581	2507	2640	1931	9622
771	158	717	148	83	53	49	46	59	171	1062	1799	2305	851	7343
771	159	854	207	159	133	99	99	74	208	955	2150	2256	900	8094
771	160	131	87	73	64	49	41	85	356	1555	2312	3131	1301	9185
771	161	201	83	59	41	37	44	64	171	888	1649	1920	850	6007
771	162	246	112	96	75	78	50	46	210	934	1636	1782	1033	6298
771	163	497	213	140	90	56	47	84	262	628	1479	1168	920	5584
771	164	269	107	67	47	27	34	24	18	847	1815	2008	551	5814
771	165	319	125	102	75	70	62	60	251	769	1622	1460	729	5644
771	166	163	69	57	48	35	48	62	278	927	1766	2021	1736	7210
771	167	279	126	87	58	49	42	58	275	833	1777	1773	766	6123
771	168	260	133	76	47	47	43	56	304	870	1802	2039	1269	6946

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MAXIMUM VOLUMES FOR PERIOD 12 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	854	213	159	133	99	99	85	356	1581	2507	3131	1931	3131	9628	40689	577

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	131	69	57	41	27	33	24	18	628	1479	1168	530	18	216	23574	

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GENERATED FLOWS FOR PERIOD 13

STA YEAR 10 11 12 1 2 3 4 5 6 7 8 9 TOTAL

771	169	245	127	75	61	58	52	85	291	764	1968	1315	571	5812
771	170	301	129	103	94	77	81	103	306	1006	1761	1820	354	6135
771	171	180	85	59	44	28	41	56	87	579	1522	1359	343	4383
771	172	297	196	103	65	65	54	87	326	692	1383	1038	351	4657
771	173	120	83	55	36	18	36	48	229	1035	1786	1892	868	6206
771	174	255	95	59	46	35	38	70	246	581	1229	1389	530	4573
771	175	373	170	133	105	117	96	65	226	876	1736	1368	494	5759
771	176	306	109	84	75	67	58	70	214	785	1614	1753	980	6115
771	177	242	98	71	39	31	40	68	172	786	1816	2028	611	6002
771	178	336	124	79	70	59	54	74	243	755	1749	1370	499	5412
771	179	212	132	116	102	82	66	99	274	964	1694	1660	670	6071
771	180	305	126	101	53	49	57	63	148	1177	1607	1491	972	6149
771	181	265	103	73	56	41	42	81	202	913	1511	1447	934	5668
771	182	383	95	63	41	34	45	94	379	974	1681	1498	738	6025

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MAXIMUM VOLUMES FOR PERIOD 13 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	383	196	133	105	117	96	103	379	1177	1968	2028	980	2028	6064	29245	470

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	120	83	55	36	18	36	48	87	579	1229	1038	343	18	276	20699	

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GENERATED FLOWS FOR PERIOD 14

STA	YEAR	10	11	12	1	2	3	4	5	6	7	8	9	TOTAL
771	183	334	110	93	76	44	40	54	140	754	1325	821	590	4381
771	184	306	142	122	86	69	64	99	338	912	1771	1662	700	6271
771	185	389	145	85	63	44	46	64	206	795	1771	1755	1253	6616
771	186	311	135	96	55	55	52	88	314	879	1974	2395	1142	7496
771	187	419	111	88	52	43	43	74	267	1046	1564	1283	853	5843
771	188	242	127	96	48	41	46	40	115	1021	1950	1752	772	6250
771	189	241	125	79	49	40	47	69	147	926	1556	1240	598	5117
771	190	274	123	71	51	49	46	53	157	697	1439	1351	849	5160
771	191	261	118	59	52	37	46	104	251	854	1407	1422	380	4991
771	192	606	246	192	152	98	92	95	392	1397	2052	2060	1152	8534
771	193	511	149	129	84	59	63	81	318	685	1533	1586	735	5933
771	194	447	223	131	93	69	63	83	264	820	1691	1320	822	6026
771	195	167	67	50	39	27	33	58	296	912	1512	1794	970	5925
771	196	208	106	70	58	42	43	46	165	967	1860	1407	1048	6020

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MAXIMUM VOLUMES FOR PERIOD 14 OF 14 YEARS OF SYNTHETIC FLOWS

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	606	246	192	152	98	92	104	392	1397	2052	2395	1253	2395	7564	31830	503

MINIMUM VOLUMES

STA	10	11	12	1	2	3	4	5	6	7	8	9	1-MO	6-MO	54-MO	AV MO
771	167	67	50	39	27	33	40	115	685	1325	821	380	27	275	22629	

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Exhibit B
Reservoir Operations

HEC-3 computer printout for the 73 cfs (47 mgd) firm-yield reservoir

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1******************************
 * RESERVOIR SYSTEM ANALYSIS  *
 * 723-X6-L2030   1 JULY 1974 *
 ******************************

                                          EAGLE RIVER WATER SUPPLY STUDY
                                           RESERVOIR OPERATION ANALYSIS
                                              GATES OPEN JUN 1-SEP 1

  NYRS   IYR    NL ICONS IDVSP IPWPR IDVPR IFLOW JUPQI
    46  1965     4     0     0     0     0     1     0

 CLOCL CFLOD IUNIT METRC    CNSTI     CNSTD      CCFS QUNIT   CACFT VUNIT IPRNT  IPRL IPWKW IUPDT IDGST
  1.00  1.00     0     0    1.000     1.000     1.000   CFS   1.000  ACFT    -1     0     0     0     0

 IRG(1)= 0  IRG(2)= 0  IRG(3)= 0  IRG(4)= 0  IRG(5)= 0  IRG(6)= 0  IRG(7)= 0  IRG(8)= 0  IRG(9)= 0  IRG(10)= 1

 NPER= 12  IPERA=  10

 PERIOD          OCT      NOV      DEC      JAN      FEB      MAR      APR      MAY      JUN      JUL      AUG      SEP
 NDAYS            31       30       31       31       28       31       30       31       30       31       31       30
 EVP            0.00     0.00     0.00     0.00     0.00     0.00     0.00     0.00     0.00     0.00     0.00     0.00

 CONTROL POINT SEQUENCE

 ********************************************
 * CP NO    1 LOWER DAMSITE                 *
 ********************************************

 MDNST  MDIV  MRES  MPWR NTSRV  IPRN  NFLW      QDV      QMN      QM2    QMXX
     2     1     1     0     0     0     0      31.      70.      0.1000000.


 RESERVOIR DATA%

  INITIAL STOR =   13300.  CEVAP = 1.000  QLKG =       0.  ISRCH =    0

                                            * * * S T O R A G E S * * *
                 OCT      NOV      DEC      JAN      FEB      MAR      APR      MAY      JUN      JUL      AUG      SEP
 LEVEL    4   39300.   39300.   39300.   39300.   39300.   39300.   39300.   39300.   13300.   13300.   39300.   39300.
 LEVEL    3   39300.   39300.   39300.   39300.   39300.   39300.   39300.   39300.   13300.   13300.   39300.   39300.
 LEVEL    2   13300.   13300.   13300.   13300.   13300.   13300.   13300.   13300.   13300.   13300.   13300.   13300.
 LEVEL    1   13300.   13300.   13300.   13300.   13300.   13300.   13300.   13300.   13300.   13300.   13300.   13300.

 STOR          1000.     2000.     4000.     8000.    12000.    15000.    21000.    27000.    35000.    50000.
 AREA          150.0     237.0     350.0     675.0     910.0    1030.0    1320.0    1690.0    2050.0    2430.0
 QCAP         10000.    20000.    30000.    40000.    50000.    60000.    80000.   100000.   150000.   200000.
 ELEV         287.00    292.80    300.00    308.00    313.00    316.00    320.30    325.10    329.40    336.10

 ********************************************
 * CP NO    2 DOWNSTREAM                    *
 ********************************************

 MDNST  MDIV  MRES  MPWR NTSRV  IPRN  NFLW      QDV      QMN      QM2    QMXX
    -1     1     0     0     0     0     1      -1.       0.      0.1000000.
 MQ AND RTIO=       1   0.000
 DIVERSION=-1.000 TIMES DIVERSION AT  1
1
OFLOW REQUIREMENTS AT  1 MULTIPLIED BY 1.019
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1ANNUAL INPUT DATA FOR 1965
 **INFLOWS
 STA   1           324.    128.     90.     67.     56.     54.     71.    243.    962.   1695.   1628.    872.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1966
 **INFLOWS
 STA   1           425.    162.     83.     51.     50.     50.     51.    235.   1150.   1858.   1958.   1177.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1967
 **INFLOWS
 STA   1           342.    103.     77.     67.     51.     53.     75.    255.   1507.   2116.   2221.   1593.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1968
 **INFLOWS
 STA   1           288.    133.     82.     72.     64.     61.     63.    356.    961.   1775.   1450.    476.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1969
 **INFLOWS
 STA   1           155.    107.     70.     39.     32.     42.     79.    322.   1252.   1564.    874.    457.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1970
 **INFLOWS
 STA   1           707.    174.    119.     99.     90.     85.     78.    218.    739.   1303.   1241.    588.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1971
 **INFLOWS
 STA   1           181.    103.     75.     57.     52.     40.     36.     82.    725.   1772.   2002.    552.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1972
 **INFLOWS
 STA   1           191.    100.     90.     65.     39.     36.     59.    145.    689.   1747.   1589.    970.
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                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1973
 **INFLOWS
 STA   1             418.    143.    123.     73.     48.     44.     71.    160.    662.   1290.   1227.    430.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1974
 **INFLOWS
 STA   1             267.     86.     54.     39.     26.     40.     77.    272.    921.   1472.   1489.   1141.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1975
 **INFLOWS
 STA   1             231.    123.     81.     48.     45.     45.     77.    313.    746.   1652.   1307.    756.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1976
 **INFLOWS
 STA   1             237.     91.     65.     56.     53.     46.     64.    177.    889.   1615.   1386.    900.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1977
 **INFLOWS
 STA   1             307.    190.    130.     94.     79.     62.     78.    249.   1333.   2120.   2424.   1100.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1978
 **INFLOWS
 STA   1             460.    158.    137.    118.     97.     79.     72.    246.    816.   1447.   1528.    971.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1979
 **INFLOWS
 STA   1             324.    116.     72.     65.     61.     69.    117.    366.   1082.   2001.   2103.   1098.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
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1ANNUAL INPUT DATA FOR 1980
 **INFLOWS
 STA   1              251.    114.     78.     40.     23.     36.     23.     35.    572.   1146.   1200.    344.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 1981
 **INFLOWS
 STA   1              179.    100.     62.     36.     21.     29.     54.    189.    673.   1367.   1246.   1054.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 1982
 **INFLOWS
 STA   1              589.    129.    100.     81.     72.     78.     83.    352.    901.   1631.   1469.    362.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 1983
 **INFLOWS
 STA   1              141.     93.     72.     51.     48.     41.     55.    205.    713.   1643.   1786.   1503.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 1984
 **INFLOWS
 STA   1              372.    142.    104.     72.     48.     47.     50.    123.    862.   1596.   1612.    639.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 1985
 **INFLOWS
 STA   1              289.    109.     49.     44.     34.     43.     77.    182.    791.   1449.   1134.    720.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 1986
 **INFLOWS
 STA   1              337.    145.     73.     66.     57.     68.     77.    325.    987.   1569.   1252.    617.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 1987
 **INFLOWS
 STA   1              214.     93.     62.     42.     24.     39.     38.     70.    784.   1728.   1409.    760.
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                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1988
 **INFLOWS
 STA   1            245.    132.    114.     89.     70.     62.     99.    295.   1159.   1907.   1646.    667.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1989
 **INFLOWS
 STA   1            181.     86.     64.     40.     28.     35.     49.    158.    678.   1690.   1240.    768.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1990
 **INFLOWS
 STA   1            242.    153.     93.     68.     59.     54.     56.    234.    782.   1434.   1214.    506.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1991
 **INFLOWS
 STA   1            163.     79.     49.     34.     20.     28.     24.     57.    605.   1237.    935.    441.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1992
 **INFLOWS
 STA   1            512.    194.     99.     75.     61.     60.     89.    253.   1347.   1873.   2000.    723.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1993
 **INFLOWS
 STA   1            135.     94.     58.     38.     33.     39.     63.    126.    730.   1595.   1363.    612.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1994
 **INFLOWS
 STA   1            246.     89.     55.     36.     26.     34.     37.     83.    579.   1506.    837.    569.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
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1ANNUAL INPUT DATA FOR 1995
 **INFLOWS
 STA   1             337.    172.    142.     88.     64.     54.     67.    184.    993.   1621.   1288.    333.


                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1996
 **INFLOWS
 STA   1             242.    113.    117.     87.     74.     69.    107.    415.   1014.   1562.   1490.    772.


                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1997
 **INFLOWS
 STA   1             436.    199.    151.    102.     62.     66.    122.    420.   1040.   1841.   1664.    822.


                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1998
 **INFLOWS
 STA   1             216.     90.     56.     35.     29.     39.     43.    155.    893.   1477.   1438.    784.


                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1999
 **INFLOWS
 STA   1             440.    205.    117.     87.     73.     83.     98.    339.    764.   1299.   1080.    900.


                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2000
 **INFLOWS
 STA   1             203.     87.     51.     36.     32.     33.     47.    204.   1675.   2121.   2743.   1885.


                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2001
 **INFLOWS
 STA   1             210.    130.     89.     69.     49.     52.     89.    349.   1226.   2084.   1621.    643.


                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2002
 **INFLOWS
 STA   1             239.     80.     59.     44.     41.     50.     59.    205.    770.   1405.   1392.    531.
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                 ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2003
 **INFLOWS
 STA   1           497.   213.   140.    90.    56.    47.    84.   262.   628.  1479.  1168.   920.

                 ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2004
 **INFLOWS
 STA   1           269.   107.    67.    47.    27.    34.    24.    18.   847.  1815.  2008.   551.

                 ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2005
 **INFLOWS
 STA   1           301.   129.   103.    94.    77.    81.   103.   306.  1006.  1761.  1820.   354.

                 ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2006
 **INFLOWS
 STA   1           180.    85.    59.    44.    28.    41.    56.    87.   579.  1522.  1359.   343.

                 ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2007
 **INFLOWS
 STA   1           297.   196.   103.    65.    65.    54.    87.   326.   692.  1383.  1038.   351.

                 ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2008
 **INFLOWS
 STA   1           120.    83.    55.    36.    18.    36.    48.   229.  1035.  1786.  1892.   868.

                 ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2009
 **INFLOWS
 STA   1           447.   223.   131.    93.    69.    63.    83.   264.   820.  1691.  1320.   822.

                 ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
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1ANNUAL INPUT DATA FOR 2010
 **INFLOWS
 STA   1            167.     67.     50.     39.     27.     33.     58.    296.    912.   1512.   1794.    970.

                          ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

 AVERAGES FOR PERIOD OF OPERATION 1965 - 2010

  1 LOWER DAMSITE

 LOC FLW     486.
 UNREG       486.
 INFLOW      486.
 REQ DIV     30.9
 DIVERSN     30.9
 SHORTGE      0.0
 EVAPO         0.

 CSV REL     104.
 RIV FLW     455.
 DES FLW     104.
 SHORTGE       0.

  2 DOWNSTREAM

 LOC FLW       0.
 UNREG       486.
 INFLOW      455.
 REQ DIV    -30.9
 DIVERSN    -30.9
 SHORTGE      0.0
 RIV FLW     486.
 DES FLW       0.
 SHORTGE       0.

 DIVERSION SHORTAGE INDEX      1  0.000     2 -1.000

 DES FLOW SHORTAGE INDEX       1  0.000     2 -1.000

 MIN FLOW SHORTAGE INDEX       1 -1.000     2 -1.000

      DIVRSION SHORTAGES  DES FLOW SHORTAGES  MIN FLOW SHORTAGES  SYS PWR SHORTAGES  AT SITE PWR SHRTGS
 STA       NO     MAX          NO     MAX          NO     MAX          NO     MAX          NO     MAX
   1        0      0.           0      0.           0      0.           0      0.           0      0.
   2        -      -            0      0.           0      0.           0      0.           0      0.

 STORAGE FREQUENCY PER 46 YEARS AT LOCATION  1
 CONS POOL       OCT     NOV     DEC     JAN     FEB     MAR     APR     MAY     JUN     JUL     AUG     SEP
 99-100 PCT       45      20       6       0       0       0       0       3      46      46      46      46
 95- 99 PCT        1      10       6       2       0       0       0       0       0       0       0       0
 90- 95 PCT        0      16      11       6       1       0       0       3       0       0       0       0
 80- 90 PCT        0       0      20      13       9       1       0       9       0       0       0       0
 70- 80 PCT        0       0       3      11      10       7       3       2       0       0       0       0
 60- 70 PCT        0       0       0      14       9      10       9       6       0       0       0       0
 40- 60 PCT        0       0       0       0      17      16      13      10       0       0       0       0
 20- 40 PCT        0       0       0       0       0      12      20       8       0       0       0       0
  1- 20 PCT        0       0       0       0       0       0       1       5       0       0       0       0
  0- 1  PCT        0       0       0       0       0       0       0       0       0       0       0       0
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EAGLE RIVER WATER SUPPLY STUDY
RESERVOIR OPERATION ANALYSIS
GATES OPEN JUN 1-SEP 1

RESERVOIR DATA

++
+ CP NO 1 LOWER DAMSITE +
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	MONTH	STORAGE	ELEV	INFLOW	OUTFLOW	EVAP	GEN PWR
		/23X AC-FT	FT	CFS	CFS	AC-FT	1000 KWH
YR 1965							
	OCT	26936.	325.05	324.	71.	0.	
	NOV	28470.	325.89	128.	71.	0.	
	DEC	27718.	325.49	90.	71.	0.	
	JAN	25552.	323.94	67.	71.	0.	
	FEB	22985.	321.89	56.	71.	0.	
	MAR	20020.	319.60	54.	71.	0.	
	APR	18162.	318.27	71.	71.	0.	
	MAY	26817.	324.95	243.	71.	0.	
	JUN	13300.	314.30	962.	1158.	0.	
	JUL	13300.	314.30	1695.	1664.	0.	
	AUG	39300.	331.32	1628.	1174.	0.	
	SEP	39300.	331.32	872.	841.	0.	
	YEAR			520.	453.	0.	
YR 1966							
	OCT	39300.	331.32	425.	394.	0.	
	NOV	39300.	331.32	162.	131.	0.	
	DEC	38118.	330.79	83.	71.	0.	
	JAN	34968.	329.38	51.	71.	0.	
	FEB	32068.	327.82	50.	71.	0.	
	MAR	28856.	326.10	50.	71.	0.	
	APR	25808.	324.15	51.	71.	0.	
	MAY	33972.	328.85	235.	71.	0.	
	JUN	13300.	314.30	1150.	1467.	0.	
	JUL	13300.	314.30	1858.	1827.	0.	
	AUG	39300.	331.32	1958.	1504.	0.	
	SEP	39300.	331.32	1177.	1146.	0.	
	YEAR			608.	577.	0.	
YR 1967							
	OCT	39300.	331.32	342.	311.	0.	
	NOV	39300.	331.32	103.	72.	0.	
	DEC	37749.	330.63	77.	71.	0.	
	JAN	35583.	329.66	67.	71.	0.	
	FEB	32738.	328.18	51.	71.	0.	
	MAR	29711.	326.56	53.	71.	0.	
	APR	28091.	325.69	75.	71.	0.	
	MAY	37485.	330.51	255.	71.	0.	
	JUN	13300.	314.30	1507.	1883.	0.	
	JUL	13300.	314.30	2116.	2085.	0.	
	AUG	39300.	331.32	2221.	1767.	0.	
	SEP	39300.	331.32	1593.	1562.	0.	
	YEAR			709.	678.	0.	
YR 1968							
	OCT	39300.	331.32	288.	257.	0.	

	NOV	39300.	331.32	133.	102.	0.
	DEC	38056.	330.77	82.	71.	0.
	JAN	36198.	329.94	72.	71.	0.
	FEB	34075.	328.90	64.	71.	0.
	MAR	31540.	327.54	61.	71.	0.
	APR	29206.	326.29	63.	71.	0.
	MAY	39300.	331.32	356.	161.	0.
	JUN	13300.	314.30	961.	1367.	0.
	JUL	13300.	314.30	1775.	1744.	0.
	AUG	39300.	331.32	1450.	996.	0.
	SEP	39300.	331.32	476.	445.	0.
	YEAR			486.	455.	0.
YR 1969						
	OCT	39300.	331.32	155.	124.	0.
	NOV	39300.	331.32	107.	76.	0.
	DEC	37318.	330.44	70.	71.	0.
	JAN	33431.	328.56	39.	71.	0.
	FEB	29531.	326.46	32.	71.	0.
	MAR	25828.	324.16	42.	71.	0.
	APR	24446.	323.06	79.	71.	0.
	MAY	37959.	330.72	322.	71.	0.
	JUN	13300.	314.30	1252.	1636.	0.
	JUL	13300.	314.30	1564.	1533.	0.
	AUG	39300.	331.32	874.	420.	0.
	SEP	39300.	331.32	457.	426.	0.
	YEAR			419.	388.	0.
YR 1970						
	OCT	39300.	331.32	707.	676.	0.
	NOV	39300.	331.32	174.	143.	0.
	DEC	39300.	331.32	119.	88.	0.
	JAN	39102.	331.23	99.	71.	0.
	FEB	38423.	330.93	90.	71.	0.
	MAR	37363.	330.46	85.	71.	0.
	APR	35922.	329.81	78.	71.	0.
	MAY	39300.	331.32	218.	132.	0.
	JUN	13300.	314.30	739.	1145.	0.
	JUL	13300.	314.30	1303.	1272.	0.
	AUG	39300.	331.32	1241.	787.	0.
	SEP	39300.	331.32	588.	557.	0.
	YEAR			457.	426.	0.
YR 1971						
	OCT	39300.	331.32	181.	150.	0.
	NOV	39300.	331.32	103.	72.	0.
	DEC	37626.	330.57	75.	71.	0.
	JAN	34845.	329.32	57.	71.	0.
	FEB	32056.	327.82	52.	71.	0.
	MAR	28230.	325.76	40.	71.	0.
	APR	24289.	322.93	36.	71.	0.
	MAY	23045.	321.94	82.	71.	0.
	JUN	13300.	314.30	725.	858.	0.
	JUL	13300.	314.30	1772.	1741.	0.
	AUG	39300.	331.32	2002.	1548.	0.
	SEP	39300.	331.32	552.	521.	0.
	YEAR			478.	447.	0.
YR 1972						
	OCT	39300.	331.32	191.	160.	0.

	NOV	39168.	331.26	100.	71.	0.
	DEC	38416.	330.93	90.	71.	0.
	JAN	36127.	329.90	65.	71.	0.
	FEB	32615.	328.12	39.	71.	0.
	MAR	28543.	325.93	36.	71.	0.
	APR	25971.	324.28	59.	71.	0.
	MAY	28601.	325.96	145.	71.	0.
	JUN	13300.	314.30	689.	915.	0.
	JUL	13300.	314.30	1747.	1716.	0.
	AUG	39300.	331.32	1589.	1135.	0.
	SEP	39300.	331.32	970.	939.	0.
	YEAR			481.	450.	0.
YR 1973						
	OCT	39300.	331.32	418.	387.	0.
	NOV	39300.	331.32	143.	112.	0.
	DEC	39300.	331.32	123.	92.	0.
	JAN	37503.	330.52	73.	71.	0.
	FEB	34491.	329.13	48.	71.	0.
	MAR	30911.	327.20	44.	71.	0.
	APR	29053.	326.20	71.	71.	0.
	MAY	32605.	328.11	160.	71.	0.
	JUN	13300.	314.30	662.	956.	0.
	JUL	13300.	314.30	1290.	1259.	0.
	AUG	39300.	331.32	1227.	773.	0.
	SEP	39300.	331.32	430.	399.	0.
	YEAR			394.	363.	0.
YR 1974						
	OCT	39300.	331.32	267.	236.	0.
	NOV	38334.	330.89	86.	71.	0.
	DEC	35369.	329.56	54.	71.	0.
	JAN	31482.	327.51	39.	71.	0.
	FEB	27248.	325.23	26.	71.	0.
	MAR	23422.	322.24	40.	71.	0.
	APR	21921.	321.04	77.	71.	0.
	MAY	32360.	327.98	272.	71.	0.
	JUN	13300.	314.30	921.	1210.	0.
	JUL	13300.	314.30	1472.	1441.	0.
	AUG	39300.	331.32	1489.	1035.	0.
	SEP	39300.	331.32	1141.	1110.	0.
	YEAR			493.	463.	0.
YR 1975						
	OCT	39300.	331.32	231.	200.	0.
	NOV	39300.	331.32	123.	92.	0.
	DEC	37995.	330.74	81.	71.	0.
	JAN	34661.	329.22	48.	71.	0.
	FEB	31482.	327.51	45.	71.	0.
	MAR	27964.	325.62	45.	71.	0.
	APR	26463.	324.67	77.	71.	0.
	MAY	39300.	331.32	313.	73.	0.
	JUN	13300.	314.30	746.	1152.	0.
	JUL	13300.	314.30	1652.	1621.	0.
	AUG	39300.	331.32	1307.	853.	0.
	SEP	39300.	331.32	756.	725.	0.
	YEAR			456.	425.	0.
YR 1976						
	OCT	39300.	331.32	237.	206.	0.

	NOV	38632.	331.02	91.	71.	0.
	DEC	36343.	330.00	65.	71.	0.
	JAN	33501.	328.59	56.	71.	0.
	FEB	30767.	327.12	53.	71.	0.
	MAR	27310.	325.27	46.	71.	0.
	APR	25035.	323.53	64.	71.	0.
	MAY	29633.	326.52	177.	71.	0.
	JUN	13300.	314.30	889.	1133.	0.
	JUL	13300.	314.30	1615.	1584.	0.
	AUG	39300.	331.32	1386.	932.	0.
	SEP	39300.	331.32	900.	869.	0.
	YEAR			468.	437.	0.
YR 1977						
	OCT	39300.	331.32	307.	276.	0.
	NOV	39300.	331.32	190.	159.	0.
	DEC	39300.	331.32	130.	99.	0.
	JAN	38794.	331.09	94.	71.	0.
	FEB	37504.	330.52	79.	71.	0.
	MAR	35031.	329.41	62.	71.	0.
	APR	33589.	328.64	78.	71.	0.
	MAY	39300.	331.32	249.	125.	0.
	JUN	13300.	314.30	1333.	1739.	0.
	JUL	13300.	314.30	2120.	2089.	0.
	AUG	39300.	331.32	2424.	1970.	0.
	SEP	39300.	331.32	1100.	1069.	0.
	YEAR			686.	655.	0.
YR 1978						
	OCT	39300.	331.32	460.	429.	0.
	NOV	39300.	331.32	158.	127.	0.
	DEC	39300.	331.32	137.	106.	0.
	JAN	39300.	331.32	118.	87.	0.
	FEB	39010.	331.19	97.	71.	0.
	MAR	37582.	330.55	79.	71.	0.
	APR	35783.	329.75	72.	71.	0.
	MAY	39300.	331.32	246.	158.	0.
	JUN	13300.	314.30	816.	1222.	0.
	JUL	13300.	314.30	1447.	1416.	0.
	AUG	39300.	331.32	1528.	1074.	0.
	SEP	39300.	331.32	971.	940.	0.
	YEAR			514.	483.	0.
YR 1979						
	OCT	39300.	331.32	324.	293.	0.
	NOV	39300.	331.32	116.	85.	0.
	DEC	37441.	330.49	72.	71.	0.
	JAN	35153.	329.47	65.	71.	0.
	FEB	32863.	328.25	61.	71.	0.
	MAR	30820.	327.15	69.	71.	0.
	APR	31699.	327.63	117.	71.	0.
	MAY	39300.	331.32	366.	211.	0.
	JUN	13300.	314.30	1082.	1488.	0.
	JUL	13300.	314.30	2001.	1970.	0.
	AUG	39300.	331.32	2103.	1649.	0.
	SEP	39300.	331.32	1098.	1067.	0.
	YEAR			628.	597.	0.
YR 1980						
	OCT	39300.	331.32	251.	220.	0.

	NOV	39300.	331.32	114.	83.	0.
	DEC	37810.	330.66	78.	71.	0.
	JAN	33984.	328.85	40.	71.	0.
	FEB	29584.	326.49	23.	71.	0.
	MAR	25512.	323.91	36.	71.	0.
	APR	20798.	320.16	23.	71.	0.
	MAY	16664.	317.19	35.	71.	0.
	JUN	13300.	314.30	572.	598.	0.
	JUL	13300.	314.30	1146.	1115.	0.
	AUG	39300.	331.32	1200.	746.	0.
	SEP	39300.	331.32	344.	313.	0.
	YEAR			325.	294.	0.
YR 1981						
	OCT	39300.	331.32	179.	148.	0.
	NOV	39168.	331.26	100.	71.	0.
	DEC	36694.	330.16	62.	71.	0.
	JAN	32622.	328.12	36.	71.	0.
	FEB	28111.	325.70	21.	71.	0.
	MAR	23609.	322.39	29.	71.	0.
	APR	20739.	320.11	54.	71.	0.
	MAY	26074.	324.36	189.	71.	0.
	JUN	13300.	314.30	673.	857.	0.
	JUL	13300.	314.30	1367.	1336.	0.
	AUG	39300.	331.32	1246.	792.	0.
	SEP	39300.	331.32	1054.	1023.	0.
	YEAR			420.	389.	0.
YR 1982						
	OCT	39300.	331.32	589.	558.	0.
	NOV	39300.	331.32	129.	98.	0.
	DEC	39163.	331.26	100.	71.	0.
	JAN	37858.	330.68	81.	71.	0.
	FEB	36179.	329.93	72.	71.	0.
	MAR	34690.	329.23	78.	71.	0.
	APR	33546.	328.62	83.	71.	0.
	MAY	39300.	331.32	352.	228.	0.
	JUN	13300.	314.30	901.	1307.	0.
	JUL	13300.	314.30	1631.	1600.	0.
	AUG	39300.	331.32	1469.	1015.	0.
	SEP	39300.	331.32	362.	331.	0.
	YEAR			492.	461.	0.
YR 1983						
	OCT	39300.	331.32	141.	110.	0.
	NOV	38751.	331.08	93.	71.	0.
	DEC	36892.	330.25	72.	71.	0.
	JAN	33743.	328.72	51.	71.	0.
	FEB	30731.	327.11	48.	71.	0.
	MAR	26967.	325.07	41.	71.	0.
	APR	24156.	322.83	55.	71.	0.
	MAY	30476.	326.97	205.	71.	0.
	JUN	13300.	314.30	713.	971.	0.
	JUL	13300.	314.30	1643.	1612.	0.
	AUG	39300.	331.32	1786.	1332.	0.
	SEP	39300.	331.32	1503.	1472.	0.
	YEAR			533.	502.	0.
YR 1984						
	OCT	39300.	331.32	372.	341.	0.

	NOV	39300.	331.32	142.	111.	0.
	DEC	39300.	331.32	104.	73.	0.
	JAN	37441.	330.49	72.	71.	0.
	FEB	34430.	329.09	48.	71.	0.
	MAR	31034.	327.27	47.	71.	0.
	APR	27927.	325.60	50.	71.	0.
	MAY	29204.	326.28	123.	71.	0.
	JUN	13300.	314.30	862.	1098.	0.
	JUL	13300.	314.30	1596.	1565.	0.
	AUG	39300.	331.32	1612.	1158.	0.
	SEP	39300.	331.32	639.	608.	0.
	YEAR			476.	445.	0.
YR 1985						
	OCT	39300.	331.32	289.	258.	0.
	NOV	39300.	331.32	109.	78.	0.
	DEC	36027.	329.86	49.	71.	0.
	JAN	32447.	328.03	44.	71.	0.
	FEB	28658.	325.99	34.	71.	0.
	MAR	25016.	323.51	43.	71.	0.
	APR	23515.	322.31	77.	71.	0.
	MAY	28420.	325.86	182.	71.	0.
	JUN	13300.	314.30	791.	1014.	0.
	JUL	13300.	314.30	1449.	1418.	0.
	AUG	39300.	331.32	1134.	680.	0.
	SEP	39300.	331.32	720.	689.	0.
	YEAR			413.	382.	0.
YR 1986						
	OCT	39300.	331.32	337.	306.	0.
	NOV	39300.	331.32	145.	114.	0.
	DEC	37503.	330.52	73.	71.	0.
	JAN	35276.	329.52	66.	71.	0.
	FEB	32764.	328.20	57.	71.	0.
	MAR	30659.	327.07	68.	71.	0.
	APR	29158.	326.26	77.	71.	0.
	MAY	39300.	331.32	325.	129.	0.
	JUN	13300.	314.30	987.	1393.	0.
	JUL	13300.	314.30	1569.	1538.	0.
	AUG	39300.	331.32	1252.	798.	0.
	SEP	39300.	331.32	617.	586.	0.
	YEAR			468.	437.	0.
YR 1987						
	OCT	39300.	331.32	214.	183.	0.
	NOV	38751.	331.08	93.	71.	0.
	DEC	36278.	329.97	62.	71.	0.
	JAN	32574.	328.10	42.	71.	0.
	FEB	28230.	325.76	24.	71.	0.
	MAR	24342.	322.97	39.	71.	0.
	APR	20521.	319.96	38.	71.	0.
	MAY	18539.	318.54	70.	71.	0.
	JUN	13300.	314.30	784.	841.	0.
	JUL	13300.	314.30	1728.	1697.	0.
	AUG	39300.	331.32	1409.	955.	0.
	SEP	39300.	331.32	760.	729.	0.
	YEAR			442.	411.	0.
YR 1988						
	OCT	39300.	331.32	245.	214.	0.

	NOV	39300.	331.32	132.	101.	0.
	DEC	39300.	331.32	114.	83.	0.
	JAN	38487.	330.96	89.	71.	0.
	FEB	36697.	330.16	70.	71.	0.
	MAR	34224.	328.98	62.	71.	0.
	APR	34032.	328.88	99.	71.	0.
	MAY	39300.	331.32	295.	178.	0.
	JUN	13300.	314.30	1159.	1565.	0.
	JUL	13300.	314.30	1907.	1876.	0.
	AUG	39300.	331.32	1646.	1192.	0.
	SEP	39300.	331.32	667.	636.	0.
	YEAR			545.	514.	0.
YR 1989						
	OCT	39300.	331.32	181.	150.	0.
	NOV	38334.	330.89	86.	71.	0.
	DEC	35984.	329.84	64.	71.	0.
	JAN	32158.	327.87	40.	71.	0.
	FEB	28036.	325.66	28.	71.	0.
	MAR	23902.	322.62	35.	71.	0.
	APR	20735.	320.11	49.	71.	0.
	MAY	24164.	322.83	158.	71.	0.
	JUN	13300.	314.30	678.	830.	0.
	JUL	13300.	314.30	1690.	1659.	0.
	AUG	39300.	331.32	1240.	786.	0.
	SEP	39300.	331.32	768.	737.	0.
	YEAR			422.	391.	0.
YR 1990						
	OCT	39300.	331.32	242.	211.	0.
	NOV	39300.	331.32	153.	122.	0.
	DEC	38733.	331.07	93.	71.	0.
	JAN	36628.	330.13	68.	71.	0.
	FEB	34228.	328.98	59.	71.	0.
	MAR	31262.	327.39	54.	71.	0.
	APR	28512.	325.91	56.	71.	0.
	MAY	36614.	330.12	234.	71.	0.
	JUN	13300.	314.30	782.	1143.	0.
	JUL	13300.	314.30	1434.	1403.	0.
	AUG	39300.	331.32	1214.	760.	0.
	SEP	39300.	331.32	506.	475.	0.
	YEAR			411.	380.	0.
YR 1991						
	OCT	39300.	331.32	163.	132.	0.
	NOV	37918.	330.70	79.	71.	0.
	DEC	34645.	329.21	49.	71.	0.
	JAN	30450.	326.95	34.	71.	0.
	FEB	25884.	324.21	20.	71.	0.
	MAR	21320.	320.56	28.	71.	0.
	APR	16665.	317.19	24.	71.	0.
	MAY	13884.	314.88	57.	71.	0.
	JUN	13300.	314.30	605.	584.	0.
	JUL	13300.	314.30	1237.	1206.	0.
	AUG	39300.	331.32	935.	481.	0.
	SEP	39300.	331.32	441.	410.	0.
	YEAR			309.	278.	0.
YR 1992						
	OCT	39300.	331.32	512.	481.	0.

	NOV	39300.	331.32	194.	163.	0.
	DEC	39102.	331.23	99.	71.	0.
	JAN	37428.	330.48	75.	71.	0.
	FEB	35138.	329.46	61.	71.	0.
	MAR	32542.	328.08	60.	71.	0.
	APR	31755.	327.66	89.	71.	0.
	MAY	39300.	331.32	253.	99.	0.
	JUN	13300.	314.30	1347.	1753.	0.
	JUL	13300.	314.30	1873.	1842.	0.
	AUG	39300.	331.32	2000.	1546.	0.
	SEP	39300.	331.32	723.	692.	0.
	YEAR			612.	581.	0.
YR 1993						
	OCT	39300.	331.32	135.	104.	0.
	NOV	38811.	331.10	94.	71.	0.
	DEC	36091.	329.89	58.	71.	0.
	JAN	32142.	327.86	38.	71.	0.
	FEB	28297.	325.80	33.	71.	0.
	MAR	24410.	323.03	39.	71.	0.
	APR	22076.	321.16	63.	71.	0.
	MAY	23538.	322.33	126.	71.	0.
	JUN	13300.	314.30	730.	871.	0.
	JUL	13300.	314.30	1595.	1564.	0.
	AUG	39300.	331.32	1363.	909.	0.
	SEP	39300.	331.32	612.	581.	0.
	YEAR			411.	380.	0.
YR 1994						
	OCT	39300.	331.32	246.	215.	0.
	NOV	38513.	330.97	89.	71.	0.
	DEC	35609.	329.67	55.	71.	0.
	JAN	31537.	327.54	36.	71.	0.
	FEB	27304.	325.26	26.	71.	0.
	MAR	23109.	321.99	34.	71.	0.
	APR	19227.	319.03	37.	71.	0.
	MAY	18045.	318.18	83.	71.	0.
	JUN	13300.	314.30	579.	628.	0.
	JUL	13300.	314.30	1506.	1475.	0.
	AUG	39300.	331.32	837.	383.	0.
	SEP	39300.	331.32	569.	538.	0.
	YEAR			344.	313.	0.
YR 1995						
	OCT	39300.	331.32	337.	306.	0.
	NOV	39300.	331.32	172.	141.	0.
	DEC	39300.	331.32	142.	111.	0.
	JAN	38425.	330.93	88.	71.	0.
	FEB	36302.	329.98	64.	71.	0.
	MAR	33337.	328.51	54.	71.	0.
	APR	31241.	327.38	67.	71.	0.
	MAY	36269.	329.97	184.	71.	0.
	JUN	13300.	314.30	993.	1348.	0.
	JUL	13300.	314.30	1621.	1590.	0.
	AUG	39300.	331.32	1288.	834.	0.
	SEP	39300.	331.32	333.	302.	0.
	YEAR			449.	418.	0.
YR 1996						
	OCT	39300.	331.32	242.	211.	0.

	NOV	39300.	331.32	113.	82.	0.
	DEC	39300.	331.32	117.	86.	0.
	JAN	38364.	330.90	87.	71.	0.
	FEB	36796.	330.20	74.	71.	0.
	MAR	34753.	329.27	69.	71.	0.
	APR	35037.	329.42	107.	71.	0.
	MAY	39300.	331.32	415.	315.	0.
	JUN	13300.	314.30	1014.	1420.	0.
	JUL	13300.	314.30	1562.	1531.	0.
	AUG	39300.	331.32	1490.	1036.	0.
	SEP	39300.	331.32	772.	741.	0.
	YEAR			509.	478.	0.
YR 1997						
	OCT	39300.	331.32	436.	405.	0.
	NOV	39300.	331.32	199.	168.	0.
	DEC	39300.	331.32	151.	120.	0.
	JAN	39286.	331.31	102.	71.	0.
	FEB	37052.	330.32	62.	71.	0.
	MAR	34825.	329.31	66.	71.	0.
	APR	36001.	329.85	122.	71.	0.
	MAY	39300.	331.32	420.	335.	0.
	JUN	13300.	314.30	1040.	1446.	0.
	JUL	13300.	314.30	1841.	1810.	0.
	AUG	39300.	331.32	1664.	1210.	0.
	SEP	39300.	331.32	822.	791.	0.
	YEAR			582.	551.	0.
YR 1998						
	OCT	39300.	331.32	216.	185.	0.
	NOV	38572.	331.00	90.	71.	0.
	DEC	35730.	329.73	56.	71.	0.
	JAN	31597.	327.57	35.	71.	0.
	FEB	27530.	325.38	29.	71.	0.
	MAR	23642.	322.41	39.	71.	0.
	APR	20118.	319.67	43.	71.	0.
	MAY	23363.	322.19	155.	71.	0.
	JUN	13300.	314.30	893.	1031.	0.
	JUL	13300.	314.30	1477.	1446.	0.
	AUG	39300.	331.32	1438.	984.	0.
	SEP	39300.	331.32	784.	753.	0.
	YEAR			441.	410.	0.
YR 1999						
	OCT	39300.	331.32	440.	409.	0.
	NOV	39300.	331.32	205.	174.	0.
	DEC	39300.	331.32	117.	86.	0.
	JAN	38364.	330.90	87.	71.	0.
	FEB	36741.	330.18	73.	71.	0.
	MAR	35558.	329.65	83.	71.	0.
	APR	35307.	329.54	98.	71.	0.
	MAY	39300.	331.32	339.	243.	0.
	JUN	13300.	314.30	764.	1170.	0.
	JUL	13300.	314.30	1299.	1268.	0.
	AUG	39300.	331.32	1080.	626.	0.
	SEP	39300.	331.32	900.	869.	0.
	YEAR			460.	429.	0.
YR 2000						
	OCT	39300.	331.32	203.	172.	0.

	NOV	38394.	330.92	87.	71.	0.
	DEC	35244.	329.51	51.	71.	0.
	JAN	31172.	327.34	36.	71.	0.
	FEB	27272.	325.25	32.	71.	0.
	MAR	23015.	321.91	33.	71.	0.
	APR	19729.	319.39	47.	71.	0.
	MAY	25987.	324.29	204.	71.	0.
	JUN	13300.	314.30	1675.	1857.	0.
	JUL	13300.	314.30	2121.	2090.	0.
	AUG	39300.	331.32	2743.	2289.	0.
	SEP	39300.	331.32	1885.	1854.	0.
	YEAR			764.	733.	0.
YR 2001						
	OCT	39300.	331.32	210.	179.	0.
	NOV	39300.	331.32	130.	99.	0.
	DEC	38487.	330.96	89.	71.	0.
	JAN	36444.	330.04	69.	71.	0.
	FEB	33488.	328.59	49.	71.	0.
	MAR	30399.	326.93	52.	71.	0.
	APR	29612.	326.50	89.	71.	0.
	MAY	39300.	331.32	349.	161.	0.
	JUN	13300.	314.30	1226.	1632.	0.
	JUL	13300.	314.30	2084.	2053.	0.
	AUG	39300.	331.32	1621.	1167.	0.
	SEP	39300.	331.32	643.	612.	0.
	YEAR			555.	524.	0.
YR 2002						
	OCT	39300.	331.32	239.	208.	0.
	NOV	37977.	330.73	80.	71.	0.
	DEC	35320.	329.54	59.	71.	0.
	JAN	31739.	327.65	44.	71.	0.
	FEB	28339.	325.82	41.	71.	0.
	MAR	25128.	323.60	50.	71.	0.
	APR	22556.	321.54	59.	71.	0.
	MAY	28875.	326.11	205.	71.	0.
	JUN	13300.	314.30	770.	1001.	0.
	JUL	13300.	314.30	1405.	1374.	0.
	AUG	39300.	331.32	1392.	938.	0.
	SEP	39300.	331.32	531.	500.	0.
	YEAR			410.	379.	0.
YR 2003						
	OCT	39300.	331.32	497.	466.	0.
	NOV	39300.	331.32	213.	182.	0.
	DEC	39300.	331.32	140.	109.	0.
	JAN	38548.	330.98	90.	71.	0.
	FEB	35981.	329.84	56.	71.	0.
	MAR	32585.	328.10	47.	71.	0.
	APR	31501.	327.52	84.	71.	0.
	MAY	39300.	331.32	262.	104.	0.
	JUN	13300.	314.30	628.	1034.	0.
	JUL	13300.	314.30	1479.	1448.	0.
	AUG	39300.	331.32	1168.	714.	0.
	SEP	39300.	331.32	920.	889.	0.
	YEAR			469.	438.	0.
YR 2004						
	OCT	39300.	331.32	269.	238.	0.

	NOV	39300.	331.32	107.	76.	0.
	DEC	37134.	330.35	67.	71.	0.
	JAN	33738.	328.72	47.	71.	0.
	FEB	29560.	326.48	27.	71.	0.
	MAR	25365.	323.79	34.	71.	0.
	APR	20711.	320.09	24.	71.	0.
	MAY	15532.	316.38	18.	71.	0.
	JUN	13300.	314.30	847.	854.	0.
	JUL	13300.	314.30	1815.	1784.	0.
	AUG	39300.	331.32	2008.	1554.	0.
	SEP	39300.	331.32	551.	520.	0.
	YEAR			489.	458.	0.
YR 2005						
	OCT	39300.	331.32	301.	270.	0.
	NOV	39300.	331.32	129.	98.	0.
	DEC	39300.	331.32	103.	72.	0.
	JAN	38794.	331.09	94.	71.	0.
	FEB	37393.	330.47	77.	71.	0.
	MAR	36088.	329.89	81.	71.	0.
	APR	36134.	329.91	103.	71.	0.
	MAY	39300.	331.32	306.	224.	0.
	JUN	13300.	314.30	1006.	1412.	0.
	JUL	13300.	314.30	1761.	1730.	0.
	AUG	39300.	331.32	1820.	1366.	0.
	SEP	39300.	331.32	354.	323.	0.
	YEAR			516.	485.	0.
YR 2006						
	OCT	39300.	331.32	180.	149.	0.
	NOV	38275.	330.86	85.	71.	0.
	DEC	35617.	329.68	59.	71.	0.
	JAN	32037.	327.81	44.	71.	0.
	FEB	27915.	325.59	28.	71.	0.
	MAR	24150.	322.82	41.	71.	0.
	APR	21399.	320.62	56.	71.	0.
	MAY	20463.	319.92	87.	71.	0.
	JUN	13300.	314.30	579.	668.	0.
	JUL	13300.	314.30	1522.	1491.	0.
	AUG	39300.	331.32	1359.	905.	0.
	SEP	39300.	331.32	343.	312.	0.
	YEAR			369.	338.	0.
YR 2007						
	OCT	39300.	331.32	297.	266.	0.
	NOV	39300.	331.32	196.	165.	0.
	DEC	39300.	331.32	103.	72.	0.
	JAN	37011.	330.30	65.	71.	0.
	FEB	34944.	329.37	65.	71.	0.
	MAR	31978.	327.78	54.	71.	0.
	APR	31072.	327.29	87.	71.	0.
	MAY	39300.	331.32	326.	161.	0.
	JUN	13300.	314.30	692.	1098.	0.
	JUL	13300.	314.30	1383.	1352.	0.
	AUG	39300.	331.32	1038.	584.	0.
	SEP	39300.	331.32	351.	320.	0.
	YEAR			391.	360.	0.
YR 2008						
	OCT	39300.	331.32	120.	89.	0.

	NOV	38156.	330.81	83.	71.	0.
	DEC	35252.	329.51	55.	71.	0.
	JAN	31180.	327.35	36.	71.	0.
	FEB	26502.	324.70	18.	71.	0.
	MAR	22430.	321.44	36.	71.	0.
	APR	19204.	319.01	48.	71.	0.
	MAY	26999.	325.10	229.	71.	0.
	JUN	13300.	314.30	1035.	1234.	0.
	JUL	13300.	314.30	1786.	1755.	0.
	AUG	39300.	331.32	1892.	1438.	0.
	SEP	39300.	331.32	868.	837.	0.
	YEAR			521.	490.	0.
YR 2009						
	OCT	39300.	331.32	447.	416.	0.
	NOV	39300.	331.32	223.	192.	0.
	DEC	39300.	331.32	131.	100.	0.
	JAN	38733.	331.07	93.	71.	0.
	FEB	36887.	330.24	69.	71.	0.
	MAR	34476.	329.12	63.	71.	0.
	APR	33331.	328.50	83.	71.	0.
	MAY	39300.	331.32	264.	136.	0.
	JUN	13300.	314.30	820.	1226.	0.
	JUL	13300.	314.30	1691.	1660.	0.
	AUG	39300.	331.32	1320.	866.	0.
	SEP	39300.	331.32	822.	791.	0.
	YEAR			506.	475.	0.
YR 2010						
	OCT	39300.	331.32	167.	136.	0.
	NOV	37204.	330.38	67.	71.	0.
	DEC	33993.	328.86	50.	71.	0.
	JAN	30105.	326.77	39.	71.	0.
	FEB	25927.	324.24	27.	71.	0.
	MAR	21671.	320.84	33.	71.	0.
	APR	19039.	318.89	58.	71.	0.
	MAY	30954.	327.23	296.	71.	0.
	JUN	13300.	314.30	912.	1178.	0.
	JUL	13300.	314.30	1512.	1481.	0.
	AUG	39300.	331.32	1794.	1340.	0.
	SEP	39300.	331.32	970.	939.	0.
	YEAR			497.	467.	0.

HEC-3 computer printout for the 108 cfs (70 mgd) firm-yield reservoir

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1*******************************
 * RESERVOIR SYSTEM ANALYSIS  *
 * 723-X6-L2030    1 JULY 1974 *
 *******************************

                                        EAGLE RIVER WATER SUPPLY STUDY
                                         RESERVOIR OPERATION ANALYSIS
                                           GATES OPEN JUN 1-AUG 1

  NYRS   IYR    NL ICONS IDVSP IPWPR IDVPR IFLOW JUPQI
    46  1965     4     0     0     0     0     1     0

 CLOCL CFLOD IUNIT METRC   CNSTI     CNSTO      CCFS QUNIT   CACFT VUNIT IPRNT  IPRL IPWKW IUPDT IDGST
  1.00  1.00     0     0   1.000     1.000     1.000   CFS   1.000  ACFT    -1     0     0     0     0

 IRG(1)= 0  IRG(2)= 0  IRG(3)= 0  IRG(4)= 0  IRG(5)= 0  IRG(6)= 0  IRG(7)= 0  IRG(8)= 0  IRG(9)= 0  IRG(10)= 1

 NPER= 12  IPERA=  10

 PERIOD           OCT      NOV      DEC      JAN      FEB      MAR      APR      MAY      JUN      JUL      AUG      SEP
 NDAYS             31       30       31       31       28       31       30       31       30       31       31       30
 EVP             0.00     0.00     0.00     0.00     0.00     0.00     0.00     0.00     0.00     0.00     0.00     0.00

 CONTROL POINT SEQUENCE

 *********************************************
 * CP NO   1 LOWER DAMSITE                    *
 *********************************************

 MDNST  MDIV  MRES  MPWR NTSRV  IPRN  NFLW      QDV      QMN      QM2     QMXX
     2     1     1     0     0     0     0      31.     100.       0.1000000.


 RESERVOIR DATA%

  INITIAL STOR =    56700.  CEVAP = 1.000  QLKG =       0.  ISRCH =    0

                                                * * * S T O R A G E S * * *
                  OCT      NOV      DEC      JAN      FEB      MAR      APR      MAY      JUN      JUL      AUG      SEP
 LEVEL   4     56700.   56700.   56700.   56700.   56700.   56700.   56700.   56700.   15700.   15700.   56700.   56700.
 LEVEL   3     56700.   56700.   56700.   56700.   56700.   56700.   56700.   56700.   15700.   15700.   56700.   56700.
 LEVEL   2     15700.   15700.   15700.   15700.   15700.   15700.   15700.   15700.   15700.   15700.   15700.   15700.
 LEVEL   1     15700.   15700.   15700.   15700.   15700.   15700.   15700.   15700.   15700.   15700.   15700.   15700.

 STOR            1000.     2000.     4000.     8000.    12000.    15000.    21000.    27000.    35000.    60000.
 AREA            150.0     237.0     350.0     675.0     910.0    1030.0    1320.0    1690.0    2050.0    2650.0
 QCAP           10000.    20000.    30000.    40000.    50000.    60000.    80000.   100000.   150000.   200000.
 ELEV           287.00    292.80    300.00    308.00    313.00    316.00    320.30    325.10    329.40    340.00

 *********************************************
 * CP NO   2 DOWNSTREAM                       *
 *********************************************

 MDNST  MDIV  MRES  MPWR NTSRV  IPRN  NFLW      QDV      QMN      QM2     QMXX
    -1     1     0     0     0     0     1      -1.       0.       0.1000000.
 MQ AND RTIO=       1   0.000
 DIVERSION=-1.000 TIMES DIVERSION AT  1
1
OFLOW REQUIREMENTS AT  1 MULTIPLIED BY 1.041
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1ANNUAL INPUT DATA FOR 1965
 **INFLOWS
 STA   1           324.   128.    90.    67.    56.    54.    71.   243.   962.  1695.  1628.   872.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1966
 **INFLOWS
 STA   1           425.   162.    83.    51.    50.    50.    51.   235.  1150.  1858.  1958.  1177.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1967
 **INFLOWS
 STA   1           342.   103.    77.    67.    51.    53.    75.   255.  1507.  2116.  2221.  1593.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1968
 **INFLOWS
 STA   1           288.   133.    82.    72.    64.    61.    63.   356.   961.  1775.  1450.   476.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1969
 **INFLOWS
 STA   1           155.   107.    70.    39.    32.    42.    79.   322.  1252.  1564.   874.   457.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1970
 **INFLOWS
 STA   1           707.   174.   119.    99.    90.    85.    78.   218.   739.  1303.  1241.   588.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1971
 **INFLOWS
 STA   1           181.   103.    75.    57.    52.    40.    36.    82.   725.  1772.  2002.   552.

                   ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1972
 **INFLOWS
 STA   1           191.   100.    90.    65.    39.    36.    59.   145.   689.  1747.  1589.   970.
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                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1973
 **INFLOWS
 STA   1             418.    143.    123.     73.     48.     44.     71.    160.    662.   1290.   1227.    430.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1974
 **INFLOWS
 STA   1             267.     86.     54.     39.     26.     40.     77.    272.    921.   1472.   1489.   1141.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1975
 **INFLOWS
 STA   1             231.    123.     81.     48.     45.     45.     77.    313.    746.   1652.   1307.    756.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1976
 **INFLOWS
 STA   1             237.     91.     65.     56.     53.     46.     64.    177.    889.   1615.   1386.    900.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1977
 **INFLOWS
 STA   1             307.    190.    130.     94.     79.     62.     78.    249.   1333.   2120.   2424.   1100.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1978
 **INFLOWS
 STA   1             460.    158.    137.    118.     97.     79.     72.    246.    816.   1447.   1528.    971.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1979
 **INFLOWS
 STA   1             324.    116.     72.     65.     61.     69.    117.    366.   1082.   2001.   2103.   1098.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
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1ANNUAL INPUT DATA FOR 1980
 **INFLOWS
 STA    1            251.    114.     78.     40.     23.     36.     23.     35.    572.   1146.   1200.    344.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1981
 **INFLOWS
 STA    1            179.    100.     62.     36.     21.     29.     54.    189.    673.   1367.   1246.   1054.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1982
 **INFLOWS
 STA    1            589.    129.    100.     81.     72.     78.     83.    352.    901.   1631.   1469.    362.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1983
 **INFLOWS
 STA    1            141.     93.     72.     51.     48.     41.     55.    205.    713.   1643.   1786.   1503.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1984
 **INFLOWS
 STA    1            372.    142.    104.     72.     48.     47.     50.    123.    862.   1596.   1612.    639.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1985
 **INFLOWS
 STA    1            289.    109.     49.     44.     34.     43.     77.    182.    791.   1449.   1134.    720.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1986
 **INFLOWS
 STA    1            337.    145.     73.     66.     57.     68.     77.    325.    987.   1569.   1252.    617.


                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1987
 **INFLOWS
 STA    1            214.     93.     62.     42.     24.     39.     38.     70.    784.   1728.   1409.    760.
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                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1988
 **INFLOWS
 STA   1          245.   132.   114.    89.    70.    62.    99.   295.  1159.  1907.  1646.   667

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1989
 **INFLOWS
 STA   1          181.    86.    64.    40.    28.    35.    49.   158.   678.  1690.  1240.   768.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1990
 **INFLOWS
 STA   1          242.   153.    93.    68.    59.    54.    56.   234.   782.  1434.  1214.   506.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1991
 **INFLOWS
 STA   1          163.    79.    49.    34.    20.    28.    24.    57.   605.  1237.   935.   441.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1992
 **INFLOWS
 STA   1          512.   194.    99.    75.    61.    60.    89.   253.  1347.  1873.  2000.   723.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1993
 **INFLOWS
 STA   1          135.    94.    58.    38.    33.    39.    63.   126.   730.  1595.  1363.   612.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 1994
 **INFLOWS
 STA   1          246.    89.    55.    36.    26.    34.    37.    83.   579.  1506.   837.   569.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
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1ANNUAL INPUT DATA FOR 1995
 **INFLOWS
 STA   1          337.   172.   142.    88.    64.    54.    67.   184.   993.  1621.  1288.   333.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 1996
 **INFLOWS
 STA   1          242.   113.   117.    87.    74.    69.   107.   415.  1014.  1562.  1490.   772.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 1997
 **INFLOWS
 STA   1          436.   199.   151.   102.    62.    66.   122.   420.  1040.  1841.  1664.   822.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 1998
 **INFLOWS
 STA   1          216.    90.    56.    35.    29.    39.    43.   155.   893.  1477.  1438.   784.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 1999
 **INFLOWS
 STA   1          440.   205.   117.    87.    73.    83.    98.   339.   764.  1299.  1080.   900.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 2000
 **INFLOWS
 STA   1          203.    87.    51.    36.    32.    33.    47.   204.  1675.  2121.  2743.  1885.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 2001
 **INFLOWS
 STA   1          210.   130.    89.    69.    49.    52.    89.   349.  1226.  2084.  1621.   643.

                         ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
1ANNUAL INPUT DATA FOR 2002
 **INFLOWS
 STA   1          239.    80.    59.    44.    41.    50.    59.   205.   770.  1405.  1392.   531.
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                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2003
 **INFLOWS
 STA   1           497.    213.    140.     90.     56.     47.     84.    262.    628.   1479.   1168.    920.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2004
 **INFLOWS
 STA   1           269.    107.     67.     47.     27.     34.     24.     18.    847.   1815.   2008.    551.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2005
 **INFLOWS
 STA   1           301.    129.    103.     94.     77.     81.    103.    306.   1006.   1761.   1820.    354.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2006
 **INFLOWS
 STA   1           180.     85.     59.     44.     28.     41.     56.     87.    579.   1522.   1359.    343.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2007
 **INFLOWS
 STA   1           297.    196.    103.     65.     65.     54.     87.    326.    692.   1383.   1038.    351.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2008
 **INFLOWS
 STA   1           120.     83.     55.     36.     18.     36.     48.    229.   1035.   1786.   1892.    868.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

1ANNUAL INPUT DATA FOR 2009
 **INFLOWS
 STA   1           447.    223.    131.     93.     69.     63.     83.    264.    820.   1691.   1320.    822.

                    ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH
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1ANNUAL INPUT DATA FOR 2010
 **INFLOWS
 STA   1             167.     67.     50.     39.     27.     33.     58.    296.    912.   1512.   1794.    970.


                                 ALL FLOWS IN  CFS, STORAGES AND EVAP IN ACFT, AND POWER IN THOUSAND KWH

 AVERAGES FOR PERIOD OF OPERATION 1965 - 2010

  1 LOWER DAMSITE

 LOC FLW     486.
 UNREG       486.
 INFLOW      486.
 REQ DIV     30.9
 DIVERSN     30.9
 SHORTGE      0.0
 EVAPO         0.

 CSV REL      71.
 RIV FLW     454.
 DES FLW      71.
 SHORTGE       0.

  2 DOWNSTREAM

 LOC FLW       0.
 UNREG       486.
 INFLOW      454.
 REQ DIV    -30.9
 DIVERSN    -30.9
 SHORTGE      0.0
 RIV FLW     185.
 DES FLW       0.
 SHORTGE       0.

 DIVERSION SHORTAGE INDEX       1  0.000      2 -1.000

 DES FLOW SHORTAGE INDEX        1  0.000      2 -1.000

 MIN FLOW SHORTAGE INDEX        1 -1.000      2 -1.000

      DIVRSION SHORTAGES  DES FLOW SHORTAGES  MIN FLOW SHORTAGES  SYS PWR SHORTAGES  AT SITE PWR SHRTGS
 STA       NO     MAX          NO     MAX          NO     MAX          NO     MAX          NO     MAX
   1        0      0.           0      0.           0      0.           0      0.           0      0.
   2        -      -            0      0.           0      0.           0      0.           0      0.

 STORAGE FREQUENCY PER 46 YEARS AT LOCATION  1
 CONS POOL       OCT     NOV     DEC     JAN     FEB     MAR     APR     MAY     JUN     JUL     AUG     SEP
 99-100 PCT       45      31      16       3       0       0       0      18      46      46      46      46
 95- 99 PCT        0      11       5       8       2       0       0       0       0       0       0       0
 90- 95 PCT        0       3       9       5       6       2       0       2       0       0       0       0
 80- 90 PCT        0       0      14      10       9       8       6       2       0       0       0       0
 70- 80 PCT        0       0       1      14       8       5       7       3       0       0       0       0
 60- 70 PCT        0       0       0       5       6       7       6       4       0       0       0       0
 40- 60 PCT        1       1       1       1      14      15      10       8       0       0       0       0
 20- 40 PCT        0       0       0       0       1       9      15       5       0       0       0       0
  1- 20 PCT        0       0       0       0       0       0       2       4       0       0       0       0
  0- 1  PCT        0       0       0       0       0       0       0       0       0       0       0       0
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EAGLE RIVER WATER SUPPLY STUDY
RESERVOIR OPERATION ANALYSIS
GATES OPEN JUN 1-AUG 1

RESERVOIR DATA

+++
+ CP NO 1 LOWER DAMSITE +
+++

/23X AC-FT FT CFSCFS AC-FT 1000 KWH

	MONTH	STORAGE	ELEV	INFLOW	OUTFLOW	EVAP	GEN PWR
YR 1965							
	OCT	56700.	338.60	324.	293.	0.	
	NOV	56284.	338.42	128.	104.	0.	
	DEC	53517.	337.25	90.	104.	0.	
	JAN	49336.	335.48	67.	104.	0.	
	FEB	44949.	333.62	56.	104.	0.	
	MAR	39968.	331.51	54.	104.	0.	
	APR	36160.	329.89	71.	104.	0.	
	MAY	42801.	332.71	243.	104.	0.	
	JUN	15700.	316.50	962.	1387.	0.	
	JUL	15700.	316.50	1695.	1664.	0.	
	AUG	56700.	338.60	1628.	930.	0.	
	SEP	56700.	338.60	872.	841.	0.	
	YEAR			520.	489.	0.	
YR 1966							
	OCT	56700.	338.60	425.	394.	0.	
	NOV	56700.	338.60	162.	131.	0.	
	DEC	53503.	337.25	83.	104.	0.	
	JAN	48338.	335.06	51.	104.	0.	
	FEB	43618.	333.05	50.	104.	0.	
	MAR	38391.	330.84	50.	104.	0.	
	APR	33393.	328.54	51.	104.	0.	
	MAY	39542.	331.33	235.	104.	0.	
	JUN	15700.	316.50	1150.	1520.	0.	
	JUL	15700.	316.50	1858.	1827.	0.	
	AUG	56700.	338.60	1958.	1260.	0.	
	SEP	56700.	338.60	1177.	1146.	0.	
	YEAR			608.	577.	0.	
YR 1967							
	OCT	56700.	338.60	342.	311.	0.	
	NOV	54796.	337.79	103.	104.	0.	
	DEC	51230.	336.28	77.	104.	0.	
	JAN	47049.	334.51	67.	104.	0.	
	FEB	42384.	332.53	51.	104.	0.	
	MAR	37342.	330.39	53.	104.	0.	
	APR	33772.	328.74	75.	104.	0.	
	MAY	41151.	332.01	255.	104.	0.	
	JUN	15700.	316.50	1507.	1904.	0.	
	JUL	15700.	316.50	2116.	2085.	0.	
	AUG	56700.	338.60	2221.	1523.	0.	
	SEP	56700.	338.60	1593.	1562.	0.	
	YEAR			709.	678.	0.	
YR 1968							
	OCT	56700.	338.60	288.	257.	0.	

	NOV	56581.	338.55	133.	104.	0.
	DEC	53323.	337.17	82.	104.	0.
	JAN	49449.	335.53	72.	104.	0.
	FEB	45506.	333.85	64.	104.	0.
	MAR	40956.	331.93	61.	104.	0.
	APR	36672.	330.11	63.	104.	0.
	MAY	50261.	335.87	356.	104.	0.
	JUN	15700.	316.50	961.	1511.	0.
	JUL	15700.	316.50	1775.	1744.	0.
	AUG	56700.	338.60	1450.	752.	0.
	SEP	56700.	338.60	476.	445.	0.
	YEAR			486.	455.	0.
YR 1969						
	OCT	56700.	338.60	155.	124.	0.
	NOV	55034.	337.89	107.	104.	0.
	DEC	51038.	336.20	70.	104.	0.
	JAN	45135.	333.70	39.	104.	0.
	FEB	39415.	331.27	32.	104.	0.
	MAR	33697.	328.70	42.	104.	0.
	APR	30365.	326.91	79.	104.	0.
	MAY	41863.	332.31	322.	104.	0.
	JUN	15700.	316.50	1252.	1661.	0.
	JUL	15700.	316.50	1564.	1533.	0.
	AUG	56700.	338.60	874.	176.	0.
	SEP	56700.	338.60	457.	426.	0.
	YEAR			419.	388.	0.
YR 1970						
	OCT	56700.	338.60	707.	676.	0.
	NOV	56700.	338.60	174.	143.	0.
	DEC	55716.	338.18	119.	104.	0.
	JAN	53503.	337.25	99.	104.	0.
	FEB	51004.	336.19	90.	104.	0.
	MAR	47930.	334.88	85.	104.	0.
	APR	44538.	333.44	78.	104.	0.
	MAY	49642.	335.61	218.	104.	0.
	JUN	15700.	316.50	739.	1279.	0.
	JUL	15700.	316.50	1303.	1272.	0.
	AUG	56700.	338.60	1241.	543.	0.
	SEP	56700.	338.60	588.	557.	0.
	YEAR			457.	426.	0.
YR 1971						
	OCT	56700.	338.60	181.	150.	0.
	NOV	54796.	337.79	103.	104.	0.
	DEC	51107.	336.23	75.	104.	0.
	JAN	46311.	334.20	57.	104.	0.
	FEB	41702.	332.24	52.	104.	0.
	MAR	35861.	329.76	40.	104.	0.
	APR	29970.	326.70	36.	104.	0.
	MAY	26711.	324.87	82.	104.	0.
	JUN	15700.	316.50	725.	879.	0.
	JUL	15700.	316.50	1772.	1741.	0.
	AUG	56700.	338.60	2002.	1304.	0.
	SEP	56700.	338.60	552.	521.	0.
	YEAR			478.	447.	0.
YR 1972						
	OCT	56700.	338.60	191.	160.	0.

NOV	54616. [?]	337.72	100.	104.	0.
DEC	51851.	336.54	90.	104.	0.
JAN	47547.	334.72	65.	104.	0.
FEB	42215.	332.46	39.	104.	0.
MAR	36128.	329.88	36.	104.	0.
APR	31606.	327.58	59.	104.	0.
MAY	32221.	327.91	145.	104.	0.
JUN	15700.	316.50	689.	936.	0.
JUL	15700.	316.50	1747.	1716.	0.
AUG	56700.	338.60	1589.	891.	0.
SEP	56700.	338.60	970.	939.	0.
YEAR			481.	450.	0.

YR 1973

OCT	56700.	338.60	418.	387.	0.
NOV	56700.	338.60	143.	112.	0.
DEC	55962.	338.29	123.	104.	0.
JAN	52150.	336.67	73.	104.	0.
FEB	47319.	334.62	48.	104.	0.
MAR	41724.	332.25	44.	104.	0.
APR	37916.	330.64	71.	104.	0.
MAY	39453.	331.29	160.	104.	0.
JUN	15700.	316.50	662.	1030.	0.
JUL	15700.	316.50	1290.	1259.	0.
AUG	56700.	338.60	1227.	529.	0.
SEP	56700.	338.60	430.	399.	0.
YEAR			394.	363.	0.

YR 1974

OCT	56700.	338.60	267.	236.	0.
NOV	53784.	337.36	86.	104.	0.
DEC	48804.	335.25	54.	104.	0.
JAN	42902.	332.75	39.	104.	0.
FEB	36848.	330.18	26.	104.	0.
MAR	31007.	327.25	40.	104.	0.
APR	27556.	325.40	77.	104.	0.
MAY	35980.	329.82	272.	104.	0.
JUN	15700.	316.50	921.	1231.	0.
JUL	15700.	316.50	1472.	1441.	0.
AUG	56700.	338.60	1489.	791.	0.
SEP	56700.	338.60	1141.	1110.	0.
YEAR			493.	463.	0.

YR 1975

OCT	56700.	338.60	231.	200.	0.
NOV	55986.	338.30	123.	104.	0.
DEC	52666.	336.89	81.	104.	0.
JAN	47317.	334.62	48.	104.	0.
FEB	42319.	332.50	45.	104.	0.
MAR	36785.	330.16	45.	104.	0.
APR	33334.	328.50	77.	104.	0.
MAY	44279.	333.33	313.	104.	0.
JUN	15700.	316.50	746.	1195.	0.
JUL	15700.	316.50	1652.	1621.	0.
AUG	56700.	338.60	1307.	609.	0.
SEP	56700.	338.60	756.	725.	0.
YEAR			456.	425.	0.

YR 1976

OCT	56700.	338.60	237.	206.	0.

	NOV	54082.	337.49	91.	104.	0.
	DEC	49778.	335.67	65.	104.	0.
	JAN	44921.	333.61	56.	104.	0.
	FEB	40367.	331.68	53.	104.	0.
	MAR	34895.	329.34	46.	104.	0.
	APR	30670.	327.07	64.	104.	0.
	MAY	33253.	328.46	177.	104.	0.
	JUN	15700.	316.50	889.	1153.	0.
	JUL	15700.	316.50	1615.	1584.	0.
	AUG	56700.	338.60	1386.	688.	0.
	SEP	56700.	338.60	900.	869.	0.
	YEAR			468.	437.	0.
YR 1977						
	OCT	56700.	338.60	307.	276.	0.
	NOV	56700.	338.60	190.	159.	0.
	DEC	56393.	338.47	130.	104.	0.
	JAN	53872.	337.40	94.	104.	0.
	FEB	50762.	336.08	79.	104.	0.
	MAR	46274.	334.18	62.	104.	0.
	APR	42882.	332.74	78.	104.	0.
	MAY	49892.	335.71	249.	104.	0.
	JUN	15700.	316.50	1333.	1877.	0.
	JUL	15700.	316.50	2120.	2089.	0.
	AUG	56700.	338.60	2424.	1726.	0.
	SEP	56700.	338.60	1100.	1069.	0.
	YEAR			686.	655.	0.
YR 1978						
	OCT	56700.	338.60	460.	429.	0.
	NOV	56700.	338.60	158.	127.	0.
	DEC	56700.	338.60	137.	106.	0.
	JAN	55655.	338.16	118.	104.	0.
	FEB	53545.	337.26	97.	104.	0.
	MAR	50102.	335.80	79.	104.	0.
	APR	46353.	334.21	72.	104.	0.
	MAY	53178.	337.11	246.	104.	0.
	JUN	15700.	316.50	816.	1415.	0.
	JUL	15700.	316.50	1447.	1416.	0.
	AUG	56700.	338.60	1528.	830.	0.
	SEP	56700.	338.60	971.	940.	0.
	YEAR			514.	483.	0.
YR 1979						
	OCT	56700.	338.60	324.	293.	0.
	NOV	55570.	338.12	116.	104.	0.
	DEC	51696.	336.48	72.	104.	0.
	JAN	47392.	334.65	65.	104.	0.
	FEB	43283.	332.91	61.	104.	0.
	MAR	39225.	331.19	69.	104.	0.
	APR	38154.	330.74	117.	104.	0.
	MAY	52357.	336.76	366.	104.	0.
	JUN	15700.	316.50	1082.	1667.	0.
	JUL	15700.	316.50	2001.	1970.	0.
	AUG	56700.	338.60	2103.	1405.	0.
	SEP	56700.	338.60	1098.	1067.	0.
	YEAR			628.	597.	0.
YR 1980						
	OCT	56700.	338.60	251.	220.	0.

	NOV	55451.	338.07	114.	104.	0.
	DEC	51946.	336.59	78.	104.	0.
	JAN	46105.	334.11	40.	104.	0.
	FEB	39885.	331.47	23.	104.	0.
	MAR	33798.	328.75	36.	104.	0.
	APR	27134.	325.17	23.	104.	0.
	MAY	20985.	320.29	35.	104.	0.
	JUN	15700.	316.50	572.	630.	0.
	JUL	15700.	316.50	1146.	1115.	0.
	AUG	56700.	338.60	1200.	502.	0.
	SEP	56700.	338.60	344.	313.	0.
	YEAR			325.	294.	0.
YR 1981						
	OCT	56700.	338.60	179.	148.	0.
	NOV	54618.	337.72	100.	104.	0.
	DEC	50129.	335.81	62.	104.	0.
	JAN	44042.	333.23	36.	104.	0.
	FEB	37711.	330.55	21.	104.	0.
	MAR	31194.	327.35	29.	104.	0.
	APR	26374.	324.60	54.	104.	0.
	MAY	29694.	326.55	189.	104.	0.
	JUN	15700.	316.50	673.	877.	0.
	JUL	15700.	316.50	1367.	1336.	0.
	AUG	56700.	338.60	1246.	548.	0.
	SEP	56700.	338.60	1054.	1023.	0.
	YEAR			420.	389.	0.
YR 1982						
	OCT	56700.	338.60	589.	558.	0.
	NOV	56343.	338.45	129.	104.	0.
	DEC	54191.	337.54	100.	104.	0.
	JAN	50871.	336.13	81.	104.	0.
	FEB	47372.	334.65	72.	104.	0.
	MAR	43868.	333.16	78.	104.	0.
	APR	40774.	331.85	83.	104.	0.
	MAY	54117.	337.51	352.	104.	0.
	JUN	15700.	316.50	901.	1516.	0.
	JUL	15700.	316.50	1631.	1600.	0.
	AUG	56700.	338.60	1469.	771.	0.
	SEP	56700.	338.60	362.	331.	0.
	YEAR			492.	461.	0.
YR 1983						
	OCT	56700.	338.60	141.	110.	0.
	NOV	54201.	337.54	93.	104.	0.
	DEC	50327.	335.90	72.	104.	0.
	JAN	45163.	333.71	51.	104.	0.
	FEB	40331.	331.66	48.	104.	0.
	MAR	34552.	329.16	41.	104.	0.
	APR	29791.	326.60	55.	104.	0.
	MAY	34096.	328.91	205.	104.	0.
	JUN	15700.	316.50	713.	991.	0.
	JUL	15700.	316.50	1643.	1612.	0.
	AUG	56700.	338.60	1786.	1088.	0.
	SEP	56700.	338.60	1503.	1472.	0.
	YEAR			533.	502.	0.
YR 1984						
	OCT	56700.	338.60	372.	341.	0.

	NOV	56700.	338.60	142.	111.	0.
	DEC	54794.	337.79	104.	104.	0.
	JAN	50921.	336.15	72.	104.	0.
	FEB	46089.	334.10	48.	104.	0.
	MAR	40678.	331.81	47.	104.	0.
	APR	35621.	329.66	50.	104.	0.
	MAY	34883.	329.34	123.	104.	0.
	JUN	15700.	316.50	862.	1153.	0.
	JUL	15700.	316.50	1596.	1565.	0.
	AUG	56700.	338.60	1612.	914.	0.
	SEP	56700.	338.60	639.	608.	0.
	YEAR			476.	445.	0.
YR 1985						
	OCT	56700.	338.60	289.	258.	0.
	NOV	55153.	337.94	109.	104.	0.
	DEC	49865.	335.70	49.	104.	0.
	JAN	44270.	333.33	44.	104.	0.
	FEB	38661.	330.95	34.	104.	0.
	MAR	33004.	328.33	43.	104.	0.
	APR	29553.	326.47	77.	104.	0.
	MAY	32443.	328.03	182.	104.	0.
	JUN	15700.	316.50	791.	1041.	0.
	JUL	15700.	316.50	1449.	1418.	0.
	AUG	56700.	338.60	1134.	436.	0.
	SEP	56700.	338.60	720.	689.	0.
	YEAR			413.	382.	0.
YR 1986						
	OCT	56700.	338.60	337.	306.	0.
	NOV	56700.	338.60	145.	114.	0.
	DEC	52888.	336.98	73.	104.	0.
	JAN	48646.	335.19	66.	104.	0.
	FEB	44314.	333.35	57.	104.	0.
	MAR	40194.	331.60	68.	104.	0.
	APR	36743.	330.14	77.	104.	0.
	MAY	48426.	335.09	325.	104.	0.
	JUN	15700.	316.50	987.	1506.	0.
	JUL	15700.	316.50	1569.	1538.	0.
	AUG	56700.	338.60	1252.	554.	0.
	SEP	56700.	338.60	617.	586.	0.
	YEAR			468.	437.	0.
YR 1987						
	OCT	56700.	338.60	214.	183.	0.
	NOV	54201.	337.54	93.	104.	0.
	DEC	49713.	335.64	62.	104.	0.
	JAN	43994.	333.21	42.	104.	0.
	FEB	37830.	330.60	24.	104.	0.
	MAR	31927.	327.75	39.	104.	0.
	APR	26156.	324.42	38.	104.	0.
	MAY	22159.	321.23	70.	104.	0.
	JUN	15700.	316.50	784.	862.	0.
	JUL	15700.	316.50	1728.	1697.	0.
	AUG	56700.	338.60	1409.	711.	0.
	SEP	56700.	338.60	760.	729.	0.
	YEAR			442.	411.	0.
YR 1988						
	OCT	56700.	338.60	245.	214.	0.

	NOV	56522.	338.53	132.	104.	0.
	DEC	55231.	337.98	114.	104.	0.
	JAN	52402.	336.78	89.	104.	0.
	FEB	48793.	335.25	70.	104.	0.
	MAR	44304.	333.34	62.	104.	0.
	APR	42162.	332.44	99.	104.	0.
	MAY	52000.	336.61	295.	104.	0.
	JUN	15700.	316.50	1159.	1738.	0.
	JUL	15700.	316.50	1907.	1876.	0.
	AUG	56700.	338.60	1646.	948.	0.
	SEP	56700.	338.60	667.	636.	0.
	YEAR			545.	514.	0.
YR 1989						
	OCT	56700.	338.60	181.	150.	0.
	NOV	53784.	337.36	86.	104.	0.
	DEC	49419.	335.51	64.	104.	0.
	JAN	43578.	333.04	40.	104.	0.
	FEB	37636.	330.52	28.	104.	0.
	MAR	31487.	327.51	35.	104.	0.
	APR	26370.	324.60	49.	104.	0.
	MAY	27784.	325.52	158.	104.	0.
	JUN	15700.	316.50	678.	850.	0.
	JUL	15700.	316.50	1690.	1659.	0.
	AUG	56700.	338.60	1240.	542.	0.
	SEP	56700.	338.60	768.	737.	0.
	YEAR			422.	391.	0.
YR 1990						
	OCT	56700.	338.60	242.	211.	0.
	NOV	56700.	338.60	153.	122.	0.
	DEC	54118.	337.51	93.	104.	0.
	JAN	49998.	335.76	68.	104.	0.
	FEB	45778.	333.97	59.	104.	0.
	MAR	40797.	331.86	54.	104.	0.
	APR	36097.	329.86	56.	104.	0.
	MAY	42184.	332.45	234.	104.	0.
	JUN	15700.	316.50	782.	1196.	0.
	JUL	15700.	316.50	1434.	1403.	0.
	AUG	56700.	338.60	1214.	516.	0.
	SEP	56700.	338.60	506.	475.	0.
	YEAR			411.	380.	0.
YR 1991						
	OCT	56700.	338.60	163.	132.	0.
	NOV	53368.	337.19	79.	104.	0.
	DEC	48080.	334.95	49.	104.	0.
	JAN	41870.	332.31	34.	104.	0.
	FEB	35484.	329.61	20.	104.	0.
	MAR	28905.	326.12	28.	104.	0.
	APR	22300.	321.34	24.	104.	0.
	MAY	17504.	317.79	57.	104.	0.
	JUN	15700.	316.50	605.	604.	0.
	JUL	15700.	316.50	1237.	1206.	0.
	AUG	56700.	338.60	935.	237.	0.
	SEP	56700.	338.60	441.	410.	0.
	YEAR			309.	278.	0.
YR 1992						
	OCT	56700.	338.60	512.	481.	0.

	NOV	56700.	338.60	194.	163.	0.
	DEC	54487.	337.66	99.	104.	0.
	JAN	50798.	336.10	75.	104.	0.
	FEB	46688.	334.36	61.	104.	0.
	MAR	42077.	332.40	60.	104.	0.
	APR	39340.	331.24	89.	104.	0.
	MAY	46595.	334.32	253.	104.	0.
	JUN	15700.	316.50	1347.	1835.	0.
	JUL	15700.	316.50	1873.	1842.	0.
	AUG	56700.	338.60	2000.	1302.	0.
	SEP	56700.	338.60	723.	692.	0.
	YEAR			612.	581.	0.
YR 1993						
	OCT	56700.	338.60	135.	104.	0.
	NOV	54261.	337.57	94.	104.	0.
	DEC	49526.	335.56	58.	104.	0.
	JAN	43562.	333.03	38.	104.	0.
	FEB	37897.	330.63	33.	104.	0.
	MAR	31995.	327.78	39.	104.	0.
	APR	27711.	325.48	63.	104.	0.
	MAY	27158.	325.18	126.	104.	0.
	JUN	15700.	316.50	730.	892.	0.
	JUL	15700.	316.50	1595.	1564.	0.
	AUG	56700.	338.60	1363.	665.	0.
	SEP	56700.	338.60	612.	581.	0.
	YEAR			411.	380.	0.
YR 1994						
	OCT	56700.	338.60	246.	215.	0.
	NOV	53963.	337.44	89.	104.	0.
	DEC	49044.	335.35	55.	104.	0.
	JAN	42957.	332.77	36.	104.	0.
	FEB	36904.	330.21	26.	104.	0.
	MAR	30694.	327.09	34.	104.	0.
	APR	24863.	323.39	37.	104.	0.
	MAY	21665.	320.83	83.	104.	0.
	JUN	15700.	316.50	579.	648.	0.
	JUL	15700.	316.50	1506.	1475.	0.
	AUG	56700.	338.60	837.	139.	0.
	SEP	56700.	338.60	569.	538.	0.
	YEAR			344.	313.	0.
YR 1995						
	OCT	56700.	338.60	337.	306.	0.
	NOV	56700.	338.60	172.	141.	0.
	DEC	56700.	338.60	142.	111.	0.
	JAN	53810.	337.38	88.	104.	0.
	FEB	49867.	335.70	64.	104.	0.
	MAR	44887.	333.59	54.	104.	0.
	APR	40841.	331.88	67.	104.	0.
	MAY	43854.	333.15	184.	104.	0.
	JUN	15700.	316.50	993.	1435.	0.
	JUL	15700.	316.50	1621.	1590.	0.
	AUG	56700.	338.60	1288.	590.	0.
	SEP	56700.	338.60	333.	302.	0.
	YEAR			449.	418.	0.
YR 1996						
	OCT	56700.	338.60	242.	211.	0.

	NOV	55391.	338.05	113.	104.	0.
	DEC	54284.	337.58	117.	104.	0.
	JAN	51333.	336.33	87.	104.	0.
	FEB	47946.	334.89	74.	104.	0.
	MAR	43888.	333.17	69.	104.	0.
	APR	42222.	332.46	107.	104.	0.
	MAY	56700.	338.60	415.	149.	0.
	JUN	15700.	316.50	1014.	1672.	0.
	JUL	15700.	316.50	1562.	1531.	0.
	AUG	56700.	338.60	1490.	792.	0.
	SEP	56700.	338.60	772.	741.	0.
	YEAR			509.	478.	0.
YR 1997						
	OCT	56700.	338.60	436.	405.	0.
	NOV	56700.	338.60	199.	168.	0.
	DEC	56700.	338.60	151.	120.	0.
	JAN	54671.	337.74	102.	104.	0.
	FEB	50617.	336.02	62.	104.	0.
	MAR	46375.	334.22	66.	104.	0.
	APR	45601.	333.89	122.	104.	0.
	MAY	56700.	338.60	420.	209.	0.
	JUN	15700.	316.50	1040.	1698.	0.
	JUL	15700.	316.50	1841.	1810.	0.
	AUG	56700.	338.60	1664.	966.	0.
	SEP	56700.	338.60	822.	791.	0.
	YEAR			582.	551.	0.
YR 1998						
	OCT	56700.	338.60	216.	185.	0.
	NOV	54023.	337.47	90.	104.	0.
	DEC	49165.	335.41	56.	104.	0.
	JAN	43017.	332.80	35.	104.	0.
	FEB	37130.	330.30	29.	104.	0.
	MAR	31227.	327.37	39.	104.	0.
	APR	25753.	324.10	43.	104.	0.
	MAY	26983.	325.09	155.	104.	0.
	JUN	15700.	316.50	893.	1052.	0.
	JUL	15700.	316.50	1477.	1446.	0.
	AUG	56700.	338.60	1438.	740.	0.
	SEP	56700.	338.60	784.	753.	0.
	YEAR			441.	410.	0.
YR 1999						
	OCT	56700.	338.60	440.	409.	0.
	NOV	56700.	338.60	205.	174.	0.
	DEC	55593.	338.13	117.	104.	0.
	JAN	52642.	336.88	87.	104.	0.
	FEB	49199.	335.42	73.	104.	0.
	MAR	46002.	334.06	83.	104.	0.
	APR	43800.	333.13	98.	104.	0.
	MAY	56344.	338.45	339.	104.	0.
	JUN	15700.	316.50	764.	1416.	0.
	JUL	15700.	316.50	1299.	1268.	0.
	AUG	56700.	338.60	1080.	382.	0.
	SEP	56700.	338.60	900.	869.	0.
	YEAR			460.	429.	0.
YR 2000						
	OCT	56700.	338.60	203.	172.	0.

	NOV	53844.	337.39	87.	104.	0.
	DEC	48679.	335.20	51.	104.	0.
	JAN	42592.	332.62	36.	104.	0.
	FEB	36872.	330.19	32.	104.	0.
	MAR	30601.	327.04	33.	104.	0.
	APR	25364.	323.79	47.	104.	0.
	MAY	29607.	326.50	204.	104.	0.
	JUN	15700.	316.50	1675.	1878.	0.
	JUL	15700.	316.50	2121.	2090.	0.
	AUG	56700.	338.60	2743.	2045.	0.
	SEP	56700.	338.60	1885.	1854.	0.
	YEAR			764.	733.	0.
YR 2001						
	OCT	56700.	338.60	210.	179.	0.
	NOV	56403.	338.47	130.	104.	0.
	DEC	53574.	337.28	89.	104.	0.
	JAN	49516.	335.55	69.	104.	0.
	FEB	44740.	333.53	49.	104.	0.
	MAR	39637.	331.37	52.	104.	0.
	APR	36900.	330.21	89.	104.	0.
	MAY	50059.	335.78	349.	104.	0.
	JUN	15700.	316.50	1226.	1773.	0.
	JUL	15700.	316.50	2084.	2053.	0.
	AUG	56700.	338.60	1621.	923.	0.
	SEP	56700.	338.60	643.	612.	0.
	YEAR			555.	524.	0.
YR 2002						
	OCT	56700.	338.60	239.	208.	0.
	NOV	53427.	337.21	80.	104.	0.
	DEC	48755.	335.23	59.	104.	0.
	JAN	43159.	332.86	44.	104.	0.
	FEB	37939.	330.65	41.	104.	0.
	MAR	32713.	328.17	50.	104.	0.
	APR	28191.	325.74	59.	104.	0.
	MAY	32495.	328.05	205.	104.	0.
	JUN	15700.	316.50	770.	1021.	0.
	JUL	15700.	316.50	1405.	1374.	0.
	AUG	56700.	338.60	1392.	694.	0.
	SEP	56700.	338.60	531.	500.	0.
	YEAR			410.	379.	0.
YR 2003						
	OCT	56700.	338.60	497.	466.	0.
	NOV	56700.	338.60	213.	182.	0.
	DEC	56700.	338.60	140.	109.	0.
	JAN	53933.	337.43	90.	104.	0.
	FEB	49546.	335.57	56.	104.	0.
	MAR	44135.	333.27	47.	104.	0.
	APR	41101.	331.99	84.	104.	0.
	MAY	48910.	335.30	262.	104.	0.
	JUN	15700.	316.50	628.	1155.	0.
	JUL	15700.	316.50	1479.	1448.	0.
	AUG	56700.	338.60	1168.	470.	0.
	SEP	56700.	338.60	920.	889.	0.
	YEAR			469.	438.	0.
YR 2004						
	OCT	56700.	338.60	269.	238.	0.

	NOV	55034.	337.89	107.	104.	0.
	DEC	50853.	336.12	67.	104.	0.
	JAN	45442.	333.83	47.	104.	0.
	FEB	39445.	331.28	27.	104.	0.
	MAR	33235.	328.45	34.	104.	0.
	APR	26630.	324.80	24.	104.	0.
	MAY	19436.	319.18	18.	104.	0.
	JUN	15700.	316.50	847.	879.	0.
	JUL	15700.	316.50	1815.	1784.	0.
	AUG	56700.	338.60	2008.	1310.	0.
	SEP	56700.	338.60	551.	520.	0.
	YEAR			489.	458.	0.
YR 2005						
	OCT	56700.	338.60	301.	270.	0.
	NOV	56343.	338.45	129.	104.	0.
	DEC	54376.	337.62	103.	104.	0.
	JAN	51855.	336.55	94.	104.	0.
	FEB	48634.	335.18	77.	104.	0.
	MAR	45314.	333.77	81.	104.	0.
	APR	43410.	332.97	103.	104.	0.
	MAY	53924.	337.42	306.	104.	0.
	JUN	15700.	316.50	1006.	1617.	0.
	JUL	15700.	316.50	1761.	1730.	0.
	AUG	56700.	338.60	1820.	1122.	0.
	SEP	56700.	338.60	354.	323.	0.
	YEAR			516.	485.	0.
YR 2006						
	OCT	56700.	338.60	180.	149.	0.
	NOV	53725.	337.34	85.	104.	0.
	DEC	49052.	335.36	59.	104.	0.
	JAN	43457.	332.99	44.	104.	0.
	FEB	37515.	330.47	28.	104.	0.
	MAR	31735.	327.65	41.	104.	0.
	APR	27034.	325.12	56.	104.	0.
	MAY	24083.	322.77	87.	104.	0.
	JUN	15700.	316.50	579.	689.	0.
	JUL	15700.	316.50	1522.	1491.	0.
	AUG	56700.	338.60	1359.	661.	0.
	SEP	56700.	338.60	343.	312.	0.
	YEAR			369.	338.	0.
YR 2007						
	OCT	56700.	338.60	297.	266.	0.
	NOV	56700.	338.60	196.	165.	0.
	DEC	54733.	337.77	103.	104.	0.
	JAN	50429.	335.94	65.	104.	0.
	FEB	46541.	334.29	65.	104.	0.
	MAR	41561.	332.18	54.	104.	0.
	APR	38705.	330.97	87.	104.	0.
	MAY	50449.	335.95	326.	104.	0.
	JUN	15700.	316.50	692.	1245.	0.
	JUL	15700.	316.50	1383.	1352.	0.
	AUG	56700.	338.60	1038.	340.	0.
	SEP	56700.	338.60	351.	320.	0.
	YEAR			391.	360.	0.
YR 2008						
	OCT	55778.	338.21	120.	104.	0.

	NOV	52684.	336.90	83.	104.	0.
	DEC	47765.	334.81	55.	104.	0.
	JAN	41678.	332.23	36.	104.	0.
	FEB	35180.	329.48	18.	104.	0.
	MAR	29093.	326.23	36.	104.	0.
	APR	23917.	322.63	48.	104.	0.
	MAY	29697.	326.55	229.	104.	0.
	JUN	15700.	316.50	1035.	1239.	0.
	JUL	15700.	316.50	1786.	1755.	0.
	AUG	56700.	338.60	1892.	1194.	0.
	SEP	56700.	338.60	868.	837.	0.
	YEAR			521.	490.	0.
YR 2009						
	OCT	56700.	338.60	447.	416.	0.
	NOV	56700.	338.60	223.	192.	0.
	DEC	56454.	338.50	131.	104.	0.
	JAN	53872.	337.40	93.	104.	0.
	FEB	50207.	335.85	69.	104.	0.
	MAR	45780.	333.97	63.	104.	0.
	APR	42686.	332.66	83.	104.	0.
	MAY	50618.	336.02	264.	104.	0.
	JUN	15700.	316.50	820.	1376.	0.
	JUL	15700.	316.50	1691.	1660.	0.
	AUG	56700.	338.60	1320.	622.	0.
	SEP	56700.	338.60	822.	791.	0.
	YEAR			506.	475.	0.
YR 2010						
	OCT	56700.	338.60	167.	136.	0.
	NOV	52654.	336.89	67.	104.	0.
	DEC	47428.	334.67	50.	104.	0.
	JAN	41525.	332.17	39.	104.	0.
	FEB	35527.	329.62	27.	104.	0.
	MAR	29256.	326.31	33.	104.	0.
	APR	24674.	323.24	58.	104.	0.
	MAY	34574.	329.17	296.	104.	0.
	JUN	15700.	316.50	912.	1198.	0.
	JUL	15700.	316.50	1512.	1481.	0.
	AUG	56700.	338.60	1794.	1096.	0.
	SEP	56700.	338.60	970.	939.	0.
	YEAR			497.	467.	0.

Exhibit C
Preliminary Seismic Evaluation

LINDVALL, RICHTER & ASSOCIATES
EARTHQUAKE SCIENCES AND ENGINEERING

PRELIMINARY SEISMIC EVALUATION

for

EAGLE RIVER DAMSITE

Near Eagle River, Alaska

July 1, 1981

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TABLES

FIGURES
(at end of report)

CONCLUSIONS

1. It is feasible to design and construct a dam at the Eagle River site, from a seismic point of view.
2. The Aleutian Trench/Arc seismic zone (plate boundary) appears to be the controlling tectonic feature at the site. It is a megathrust that underlies the site at about 20 miles depth. It is part of the Circum-Pacific Seismic Belt, and as such, could conceivably cause a maximum credible Richter magnitude earthquake of 8.7.
3. Smaller faults closer to the site (Eagle River Thrust, Knik and Castle Mountain) are not known to be as active nor as significant in terms of seismic design.
4. The seismic design of the dam and appurtenant works should incorporate a 0.4 peak acceleration (see Table 2). Effective peak g would be less, but not less than 0.33 g.
5. Lindvall, Richter & Associates were not asked to provide digitized time histories, ground motions, or site-specific response spectra in this preliminary report. Likewise we were not asked to discuss reservoir-induced seismicity nor liquefaction potential. We would be happy to provide such information if desired.

TECTONIC SETTING

There are two principal earthquake zones in Alaska. These two zones are part of the seismic belt which rings the Pacific Ocean (Gutenberg and Richter, 1954). One, the Denali fault zone, is generally considered to begin north of Yakutat Bay off the Gulf of Alaska and to extend southeastward to the west coast of Vancouver Island. Because it lies 120 miles distant from the Eagle River damsite, it will not be discussed further. The other, the Aleutian Trench, is described as following the arc of the Aleutian Islands from their western extremity through the Kenai Peninsula to east of Prince William Sound; it is one of the world's most active seismic zones (Plafker, 1971; St. Amand, 1957; and Figs. 1, 2, and 3 herein). It was near the intersection of these two faults that the 1964 Good Friday earthquake occurred. Fortunately, the previous 7 earthquakes of Richter magnitude 8 or larger that have struck southeast Alaska since 1899 occurred in sparsely populated areas.

The historic record going back to the Russian Settlement of 1784 (Sykes et al., 1980), suggests great earthquakes within the Aleutian Trench/Arc of the Circum-Pacific Seismic Belt at relatively short intervals, so that the entire trench/arc is involved. Seismic gaps (areas of accumulating strain energy) are eventually plugged. Thus, probably any given locality within the trench/arc zone is subject to a great earthquake at least once every few hundred years.

The trench is a shallow-dipping subduction zone (Figs. 4, 5, and 6), as revealed by instrumentally recorded aftershock patterns and from the focal depths of several moderate to large earthquakes, including the 1964 event. This means that the megathrust -- the source of great earthquakes -- is as close as 20 miles in a vertical direction, and slightly farther in a horizontal direction. The implication here is that the common design criterion of vertical g equals 75 percent of horizontal g is not necessarily valid; vertical and horizontal g should be approximately equal for the Eagle River site.

HISTORICAL SEISMICITY

General

South-central Alaska, including the eastern Aleutian Trench/Arc, is one of the most active regions in the world. Past large earthquakes suggest a Richter M 8 or greater event every ten years, on the average. Any given area within this region, however, may escape extensive damage from a large quake for more than a century, as discussed later.

The following list of large earthquakes is representative; those of 1788-1880 are adapted from Sykes et al. (1980).

Table 1. List of Large Earthquakes in Alaska (M >7.2) 1784-1980

Date		Location		Richter M
1788	July 22	55N	160W?	8+
1788	Aug. 7	55N	160W?	7-8?
1792	---	57N	152W?	7-8?
1844	---	57N	152W?	7-8?
1847-48	---	55N	160W?	8+
1848	---	58N	137W?	7-8?
1854	---	57N	152W?	7-8?
1878	Aug. 29	57N	152W?	7-8?
1880	---	55N	155W?	7-8?
1896	May --	61N	144W?	7-8?
1899	July 14	55N	160W	7.7

Table 1. (contd)

List of Large Earthquakes in Alaska (M >7.2) 1784-1980

1899	Sep. 4	60N	142W	8.3
1899	Sep. 10	60N	140W	7.8 (foreshock)
1899	Sep. 10	60N	140W	8.6
1900	Oct. 9	60N	142W	8.3
1901	Dec. 31	52N	177W	7.8
1902	Jan. 1	55N	165W	7.8
1903	June 2	57N	157W	8.3
1904	Aug. 27	64N	151W	8.3
1905	Feb. 14	53N	178W	7.9
1906	Aug. 17	51N	179E	8.3
1907	Sep. 2	52N	173E	7-3/4
1929	Mar. 7	51N	170W	8.6
1929	July 7	52N	178W	7.3
1929	Dec. 17	52½N	171½E	7.6
1937	July 22	64-3/4N	146-3/4W	7.3
1937	Sep. 3	52½N	177½W	7.3
1938	Nov. 10	55.5N	158.4W	8.7
1940	July 14	51.8N	177.5E	7.8
1943	Nov. 3	61.7N	151W	7.3
1947	Apr. 1	52.3N	163.2W	7.4
1948	May 14	54.7N	161W	7.5
1949	Aug. 22	53.6N	133.3W	8.1*
1957	Mar. 9	51.6N	175.4W	8.2
1957	Mar. 12	51.4N	176.9W	7.3

Table 1. (contd)

List of Large Earthquakes in Alaska (M >7.2) 1784-1980

1957	Apr. 19	52.2N	166.3W	7.3
1958	Apr. 7	66.0N	156.6W	7.3
1964	Mar. 28	61.1N	147.6W	8.5**
1965	Feb. 4	51.3N	178.6E	7.9
1965	Mar. 30	50.6N	177.9E	7.5
1972	July 30	58N	136W	7.6
1979	Feb. 28	60.6N	141.6W	7.4

*Actually, British Columbia (Queen Charlotte Is.)

**Revised upward from M 8.4 by C. F. Richter from later data reduction.

Good Friday Earthquake of 1964

The earthquake that struck at 5:36 P.M. on March 27, 1964, originating an estimated ten to twenty miles below the earth's surface, had a magnitude of 8.5, established by C. F. Richter, the U.S. Coast and Geodetic Survey, and the U.S. Geological Survey. It released somewhat more energy than the 1906 San Francisco quake. Significant crustal deformation occurred (National Academy of Sciences, 1971, 1972, 1973, and Fig. 7 herein).

Kelleher and Savino (1975) reviewed the records of past earthquakes in this region. They found a seismically quiet period from 1944 to 1954; from 1954 to 1964 there was increased seismic activity near the rupture zone. Lay and Kanamori (in preparation; personal communication, 1981) suggest that large quakes in the eastern part of the Alaska peninsula can result in 800 km rupture lengths.

Eagle River is some 60 miles from the Good Friday epicenter and focus, and as a result, experienced greatly attenuated intensities -- on the order of MM VII. Had the epicenter been closer to the site, as is possible in future events, the intensity and the ground accelerations could have been severe.

MAJOR FAULT IN REGION

Aleutian Trench/Arc

The maximum credible earthquake for the entire region can be reasonably associated with the Aleutian Trench/Arc, an active boundary between two crustal plates of very large dimensions. Indeed, it is appropriate to regard this zone as a major part of the Circum-Pacific Seismic Belt, and to regard the other active faults of south-central Alaska as lesser-order expressions of this broader fault system. Appraisal of tectonic activity on this basis leads to results consonant with those derived through application of the criteria listed for the maximum credible earthquakes.

Because this is the largest and most active fault zone in the region, the necessity of designating a local or near-field event on a closer but smaller fault is not necessary. Effective ground accelerations resulting from the smaller faults could be similar to that of the Aleutian Trench/Arc, but the duration of shaking would be shorter.

LOCAL FAULTS IN PROJECT AREA

Eagle River Thrust

The Eagle River Thrust forms a sinuous trace through the Chugach Mountains (Clark, 1973). It passes within a mile of the proposed damsite. It has apparently been offset by high angle cross faults. Recent activity on this thrust fault is not proven, but is not as critical for project feasibility or for design purposes as the Aleutian Trench/Arc structure, because effective ground accelerations at the site would be comparable but duration of shaking resulting from large event on the Trench/Arc structure would be much longer.

Knik Fault

The existence of the Knik fault was inferred only until recently. Its total length is not known, but it exceeds 150 miles. It passes within two miles of the proposed damsite. Recent activity on this fault is not proven, but it is not as critical for project feasibility or design purposes as is the Aleutian Trench/Arc. Effective ground accelerations would be in the same range for events on both faults but the longer duration of shaking generated by the Trench/Arc structure makes it the governing event.

Castle Mountain Fault

The Castle Mountain (Lake Clark) fault extends for more than 300 miles, and is 25 miles from the damsite at its closest point. It is considered an active fault, but is not as critical for project feasibility or design purposes as the Aleutian Trench/Arc, because of lesser effective accelerations and shorter duration of shaking.

SEISMICITY

State of the Art

Prior to the San Fernando, California, earthquake of 1971, the worldwide distribution of strong-motion seismographs was such that any earthquake that occurred triggered no more than a few such instruments, and the great majority of earthquakes went completely unrecorded by instruments of low enough magnification to provide the desired information. Up to 1971 the library of recorded strong ground motions included records obtained from events ranging widely in magnitude and rupture mechanisms, at a variety of distances and on many different types of foundation material. No more than two or three records on various foundations and at different distances represented any one earthquake. Of this library only a few records showed strong ground motion of 0.2 gravity acceleration or higher; none were obtained from earthquakes of Richter Magnitude greater than 7.7 (such as the 1952 Kern County event).

It is not surprising, therefore, that the correlations which were attempted among the variables such as the magnitude of the event, the epicentral (or other) distance, the foundation conditions, and the peak acceleration or spectrum intensity exhibited extreme scatter.

In the San Fernando magnitude 6.4 earthquake of February 1971 strong motion records were obtained from more than two hundred instruments, so that, for the first time, patterns of behavior could be sought. Similarly many records were obtained from the 1979 Imperial Valley earthquake of M 6.6. However, even in these circumstances, subsequent analyses exhibit a large diversity of response. Apparently the diverse nature of the source motions and the mechanisms of wave propagation through the varied materials of southern California are extremely complex phenomena. Even the large number of stations at which records were obtained in these earthquakes was not sufficient to enable unequivocal relations to be derived, such as the effect of the recording site properties on the accelerations measured. General indications of the attenuation of acceleration amplitudes with distance, however, have been obtained, albeit still with a substantial scatter; different attenuation along different ray paths is also evident.

From the point of view, therefore, of establishing a design earthquake at a particular site, a large number of uncertainties are still present. The peak acceleration may vary by a factor of two. The ground motion selected for particular conditions by different analysts will vary widely, since elements of personal judgment play a large part in the selection process. In particular, instrumental evidence for the strong-motion behavior of the ground during very large earthquakes, and at close distances to the rupture zone for all earthquake magnitudes, is essentially entirely lacking.

It must therefore be pointed out that, whereas the method adopted in this report of arriving at earthquake ground motions at the Eagle River site is felt to be reasonable, conservative, and in accordance with the current state of practice for similar calculations, there is no guarantee that the ground motion parameters which might occur at the site during some future earthquake will correspond precisely to those assumed for use in the dam performance calculations. The previous history of earthquake studies demonstrates that each significant earthquake has appreciably modified existing concepts of both the mechanisms and magnitudes of effects involved. The same procedural consequences will probably follow each future event, especially if it is accompanied by ground motions recorded at many stations.

Maximum Credible Earthquakes

The magnitude of the maximum credible earthquake for each fault (Table 2) has been estimated on the basis of the following factors:

A. Historic record of seismicity.

B. Fault length.

C. Maximum rupture length for a single seismic event in historic time.

D. Maximum surface displacement for a single seismic event in historic time.

E. Behavior of the fault during the past 3 to 5 million years, including distribution and dominant style of movement, cumulative separation or displacement and apparent average rate of slip.

F. Inferred role of the fault in the regional tectonic framework during the past 3 to 5 million years.

The maximum <u>expectable or probable</u> earthquake magnitude, in this highly seismic region, would be only one- or two-tenths of a magnitude less than the maximum credible earthquake.

Maximum Credible Bedrock Accelerations

The peak accelerations shown on Table 2 were derived from judgmental collation of the sources listed plus the judgment derived from many years of experience in this field.

Although peak horizontal accelerations were asked for in this report, we recommend effective g (two-thirds peak) be used in design, and such should not be less than 0.33 g,

Table 2. Seismic Parameters for Eagle River Damsite, Alaska

Fault	Total length (mi)	Horizontal distance to site (mi)	Maximum credible Richter M	Site peak g	Duration strong shaking (seconds)	Probability of occurrence next 100 years
Aleutian Trench/Arc	>2000	30-200 20*	8.7	0.35 0.4*	60+	High
Denali	>1000	120	8.4	0.05	60+	High
Eagle River Thrust	>100	1 (horiz. & vert.)	7.0	0.7	20	Moderate-Low
Knik	>150	2	7.5	0.7	35	Moderate-Low
Castle Mountain	>300	25	7.8	0.25	35	Moderate-High

*Vertical distance; vertical acceleration (see text)

References

Lindvall, Richter & Associates (this report)
Schnabel and Seed (1973)
Housner (1969)
Housner and Jennings (1973)
Page et al. (1972)
Hudson et al. (1971)
Hudson and Cloud (1972)
Richter (1958)
Von Huene (1972)
Sykes et al. (1980)

as also shown by Algermissen and Perkins (1976), Fig. 8 herein. (Newmark and Hall, 1973, suggest 0.33 g for the Alyeska Pipeline design, along with 16 in/sec design velocity for structures.) Peak horizontal g usually represents an isolated spike with a duration so short that large structures do not respond. Digitized time histories would better define effective ground motions, but we were not asked to provide these in this preliminary report.

As discussed previously under Aleutian Trench/Arc, both horizontal and vertical g should be approximately the same for design purposes.

A variety of studies has been made relating parameters describing ground motion at a site to its distance from a causative fault or earthquake epicenter (e.g. Schnabel and Seed, 1973; Housner and Jennings, 1973; McGuire, 1974). For earthquakes greater than magnitude 8 originating at distances of the order of 20-30 miles, investigations indicate that peak accelerations of about 0.4 g will be experienced at the site, and that the duration of strong ground motions may exceed 60 seconds.

It was considered that the second worst case of ground motion at the site would result from the initiation of a fault rupture at a point on the fault some intermediate distance from the site. With a rupture velocity of a few kilometers a second, a rupture initiating, say 100 miles away, would reach the stretch of the fault closest to the site in perhaps 30 seconds, pass by, and terminate at a point on the fault some distance away. Because of the Doppler effect as the rupture moves towards the site, the early stages would be characterized by fairly high frequency shaking, attenuated to some extent by distance. The peak intensity and frequency of shaking would occur at the point of closest approach relatively little diminished by attenuation through the ground. Ultimately as the rupture continued at increasing distances, the intensity of shaking would fall off, to form the tail of the hypothetical record.

No ground motion has yet been recorded in conditions matching these requirements, and it is necessary to construct an artificial acceleration history to best represent the circumstances of site and hypothetical earthquake. The largest earthquake for which a strong-motion record has been obtained is still the 1952 Kern County, California, event, for which records of three components were obtained at Taft.

Earthquake Recurrence Probabilities

As is true for most areas of the world, the historical record is simply too short to make meaningful extrapolations into the future, except for specific faults at specific localities. Alaska is not one of these favored places having a long written record.

However, Sykes et al. (1980) have dug into the records of the period of Russian settlement in Alaska, 1784-1867. They have interpreted past major events based on the early descriptions of damage and physical changes. Adaptations from this record are shown on Table 1. This record, combined with the record of this century, has led Sykes et al. (1980) to suggest a recurrence interval of great earthquakes in the central Aleutian Arc at 50-75 years. McCann et al. (1980) suggest a recurrence interval of 80 years for the Yakataga area; Kanamori (1977a) suggests 220 years recurrence interval for a repeat of the 1964 event at Prince William Sound.

Elaborate statistical investigations are often applied to draw conclusions about the probable magnitude and frequency of future earthquakes in a given area. Such work adds very little to estimates of risk which may be required prior to establishing a design earthquake. Simpler common-sense evaluation of historical and geological data is desirable; such data, as well as seismographic catalogs, are grossly incomplete from the statistical point of view.

Kelleher (1970) indicates a seismic gap (accumulation of strain energy) southwest of the 1964 epicenter, and predicts an earthquake of lesser magnitude than 1964 prior to year 2000. Sykes (1971) suggests a seismic gap exists east of the 1964 epicenter. Either prediction could yield equal or lesser intensities of shaking at the Eagle River site to those caused by the 1964 earthquake. However, after a period of tens of years, the seismic gap falls again back to the Prince William Sound area -- the closest area to Eagle River that has experienced a great earthquake in recorded history.

In using even the most complete catalogs of small events to infer the probability of large ones, it is assumed that where large earthquakes occur occasionally, smaller ones occur progressively more frequently, (e.g. Shakal and Willis, 1972; Marachi and Dixon, 1972). This assumption is partially justified, as there appears to be a real relationship of this character for many faults. For the globe as a whole, and for certain local but usually rather large regions, it is commonly put in the form: $\log N = a + bM$, where "N" is the number of earthquakes with magnitudes near a given magnitude "M", during an average time interval of a year, while "a" and "b" are constants; "b" is necessarily a negative number that usually turns out to be near -2, often about -1.8.

It is commonly overlooked, however, that this is a statistically general relation only, and that it is subject to many local and temporary exceptions. Certain faults and segments of faults that occasionally are the seats of large earthquakes (e.g., those segments of the San Andreas fault on which the earthquakes of 1857 and 1906 originated) often exhibit intervals of quiescence, during which even small events are infrequent.

The occurrence of small earthquakes is not very useful for estimating earthquake risk in a given locality, unless their epicenters establish the activity of a nearby fault. It is our judgment that formulation of design earthquakes to apply at a given place should not be based upon elaborate estimates of probabilities, which at best are derived from fragmentary and questionable history and statistics. Instead, the few larger known historical earthquakes, together with geological data on active faults, should be used to envision seismic occurrences likely to affect a structure at the locality of concern.

In the past century, enough moderate and large earthquakes have occurred in Alaska to define the Aleutian Trench/Arc, the Denali, and some other faults as active (Brogan, 1975). Small earthquakes may be random near-surface crustal adjustments, unrelated to recognized faults in the vicinity. Thus, until a given fault ruptures in an earthquake or has displaced geologically recent deposits, its potential for future activity cannot definitely be known. Such is the case for the Eagle River and Knik faults. If their earthquake return periods were only 300 years, we would not know it because alluviation and erosion would have destroyed surface displacements, and no one was in the region to record a past event. Also, the habit of some faults, or segments of faults, may be to cause large earthquakes almost exclusively, and almost never cause small or moderate earthquakes (e.g. San Andreas fault). For these reasons we do not place strong reliance on most graduated magnitude versus recurrence interval graphs.

Therefore, it would be prudent for the entire Eagle River dam installation to be designed to expect, at a minimum, a repeat of the 1964 earthquake, at a somewhat closer proximity, during its useful life.

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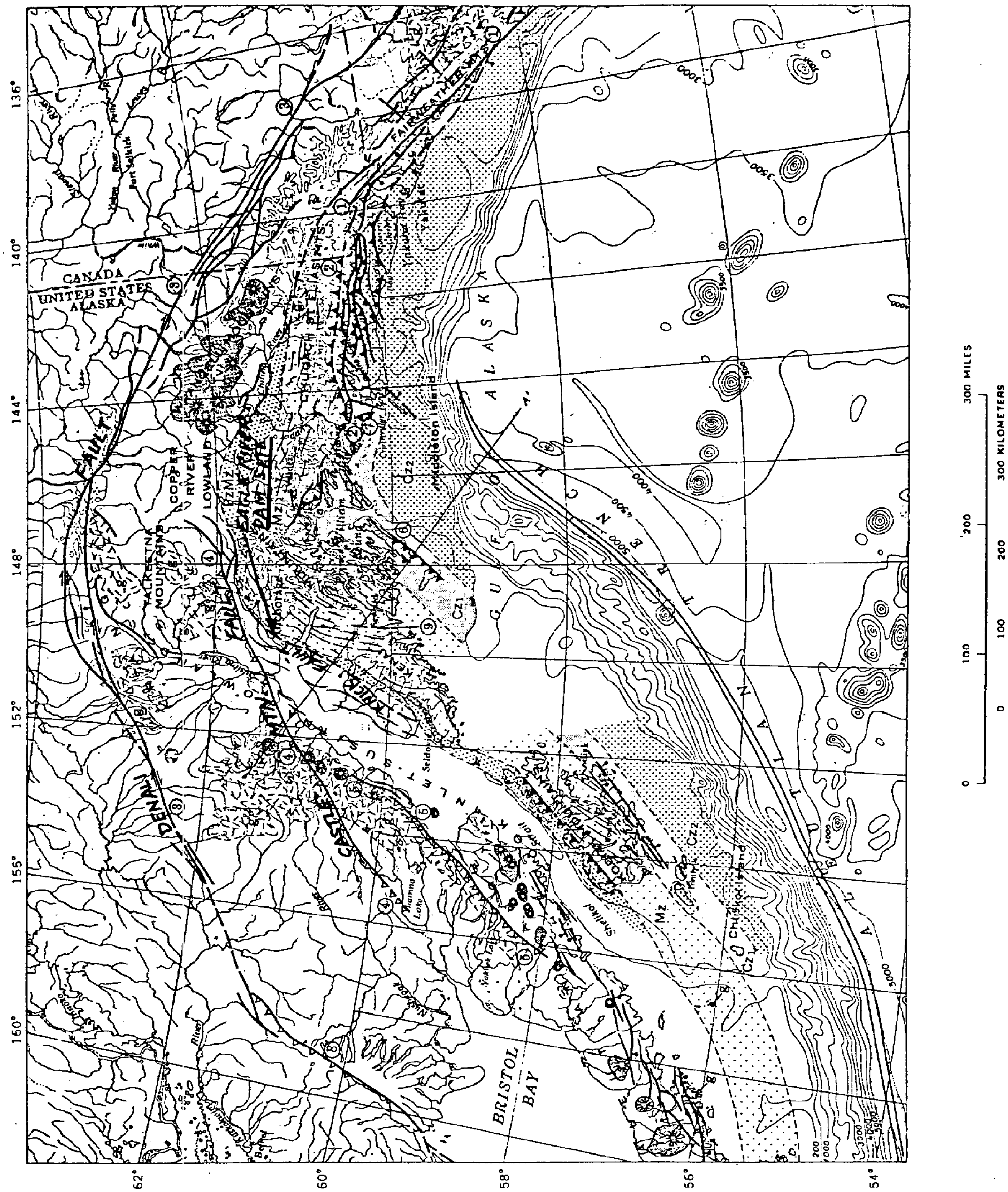


Fig. 1. Generalized fault map of southeast Alaska.

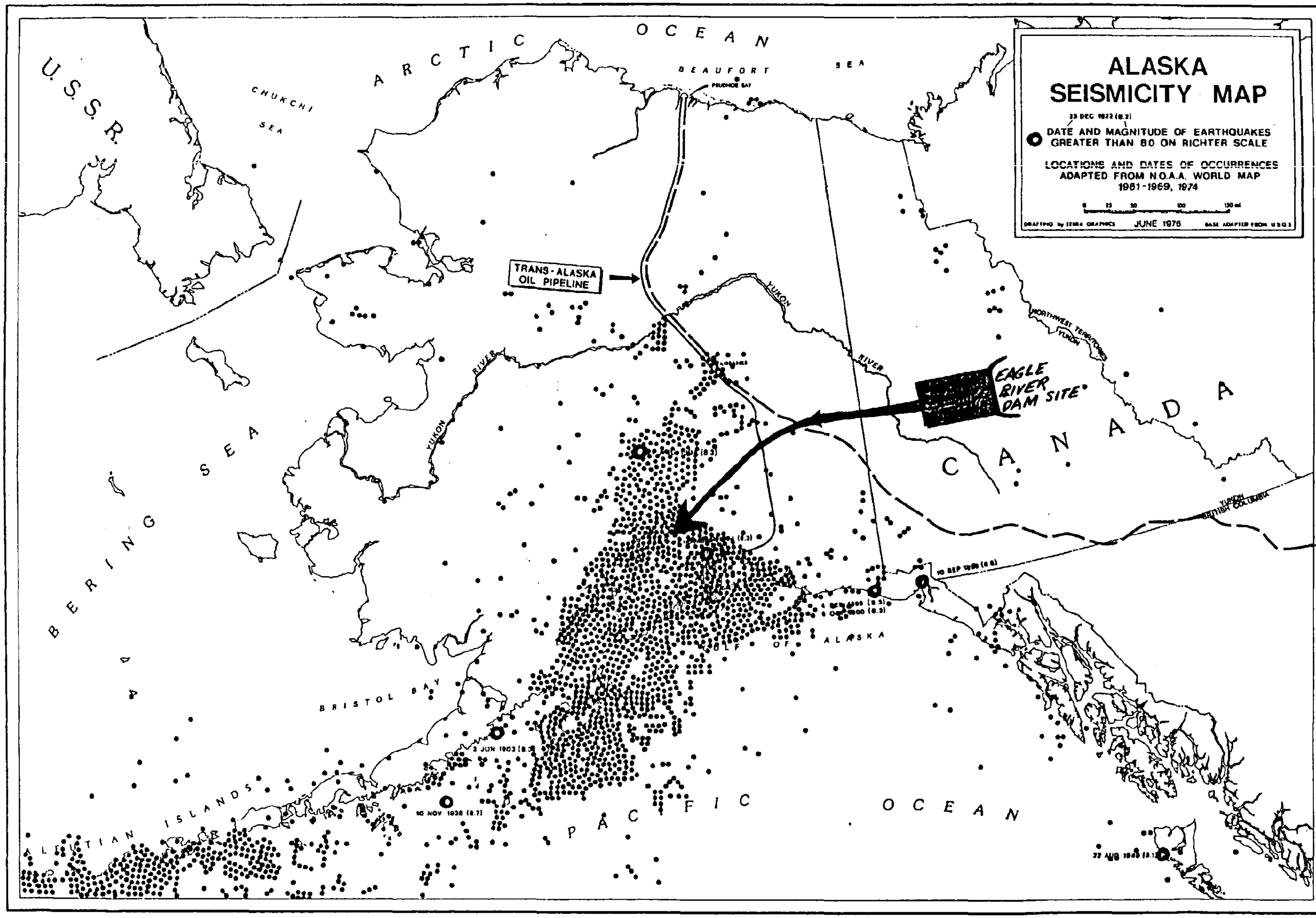
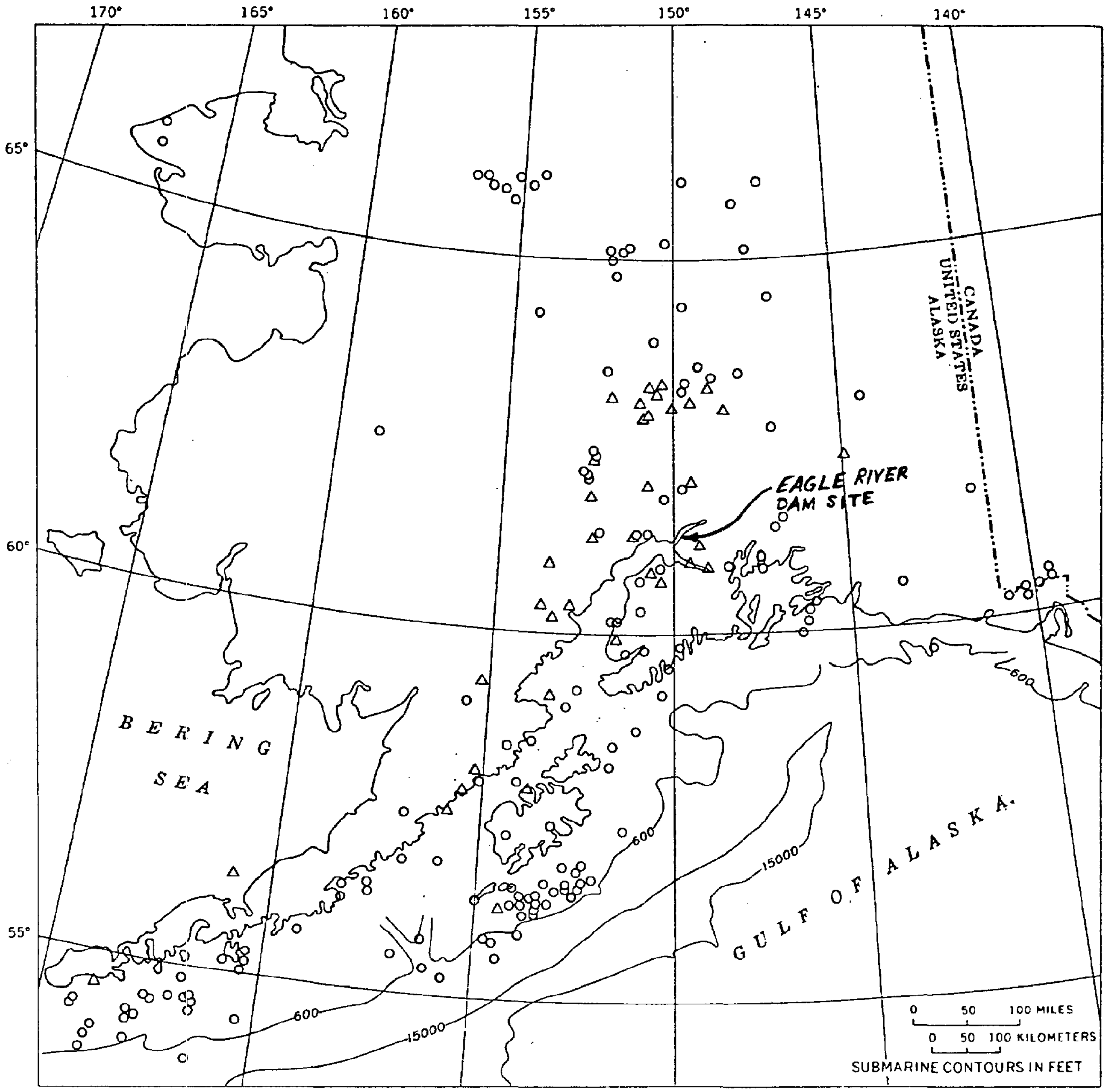
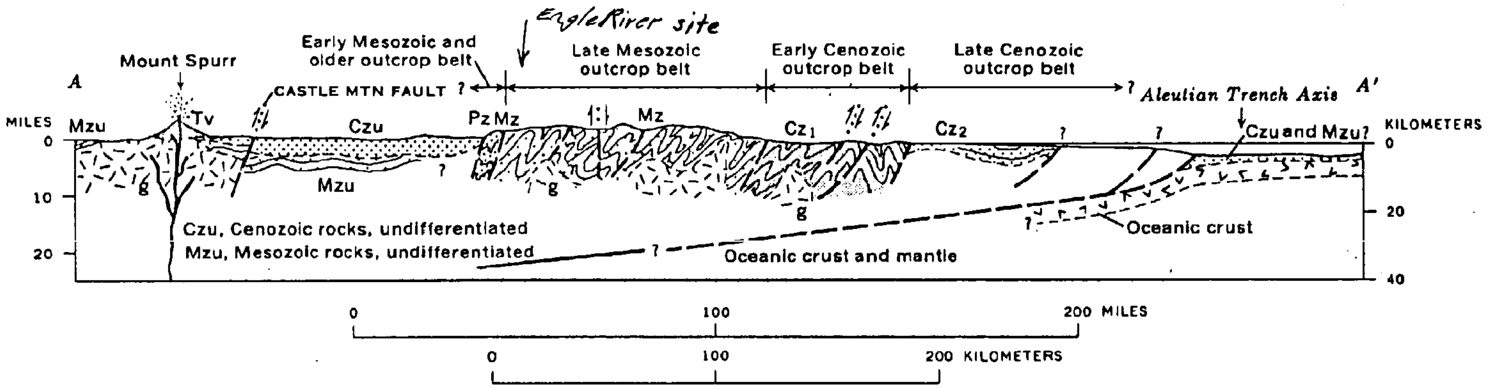

ALASKA
SEISMICITY MAP
23 DEC 1972 (8.2)
DATE AND MAGNITUDE OF EARTHQUAKES GREATER THAN 8.0 ON RICHTER SCALE
LOCATIONS AND DATES OF OCCURRENCES ADAPTED FROM N.O.A.A. WORLD MAP 1961-1969, 1974
JUNE 1976
TRANS-ALASKA OIL PIPELINE
EAGLE RIVER DAM SITE
ARCTIC OCEAN
BEAUFORT SEA
CHUKCHI SEA
U.S.S.R.
CANADA
BERING SEA
BRISTOL BAY
GULF OF ALASKA
PACIFIC OCEAN
ALEUTIAN ISLANDS
YUKON RIVER
2 JUN 1903 (8.3)
10 NOV 1938 (8.7)
10 SEP 1899 (8.6)

Fig. 2



30.—Epicenters of earthquakes ($M \geq 4$) in central Alaska during the period January 1954 to March 1963. Shallow depth ($\leq$ 70 km) earthquakes indicated by circles; intermediate depth ($\geq$ 70 km) indicated by triangles. Data after Tobin and Sykes (1966).

Fig. 3



EXPLANATION

Tv — Andesitic extrusive rocks of active or dormant volcanoes

Cz2 — Late Cenozoic bedded rocks
Lighter pattern where projected offshore

Cz1 — Early Cenozoic bedded rocks
Lighter pattern where projected offshore

Mz — Late Mesozoic bedded rocks
Lighter pattern where projected offshore

PzMz — Paleozoic and early Mesozoic bedded rocks
Lighter pattern where projected offshore

g — Granitic plutonic rocks

Undifferentiated rocks

Approximate contact
Includes possible fault contacts. Dashed where inferred or concealed

Thrust or reverse fault
Dashed where inferred. Sawteeth on upper plate. Open teeth indicate major fault

Steeply dipping fault
Dashed where inferred. Arrows indicate relative lateral displacement; bar and ball on relatively downthrown side

Trend lines showing strike of bedding, schistosity, and folds

Major faults and faults with known Holocene movement Asterisk indicates known Holocene movement; double asterisk indicates historic movement		
No	Fault	Data Source
1**	Fairweather	Tocher (1960); Tarr and Martin (1912); Plafker (1967)
2.	Chugach—St Elias (probable Holocene movement)	Miller and others (1959, p. 42); Plafker (1967)
3*	Denali	St. Amand (1957); Hamilton and Myers(1966); Grantz (1966)
4*	Castle Mtn-Lake Clark	Martin and Katz (1912, p. 72–75); Kelly (1963, p. 289); Grantz (1965, sheet 3)
5.	Bruin Bay	Burk (1965, p. 139); R. L. Detterman, (oral commun., 1967)
6**	Patton Bay and Hanning Bay	Plafker (1968)
7*	Ragged Mtn	Miller (1961)
8*	Holitna-Togiak	Hoare (1961, p. 608–610)
9.	Kenai lineament (possible 1964 movement)	This paper

Fig. 4

20.—~~Generalized tectonic map and~~ idealized vertical section showing selected rock units and structural features of south-central Alaska. Indicated displacement direction on faults is the net late Cenozoic movement only. Geology modified from a manuscript tectonic map of Alaska by P. B. King and from unpublished U.S. Geological Survey data; the thickness of crustal layers and the structure shown in the section are largely hypothetical.

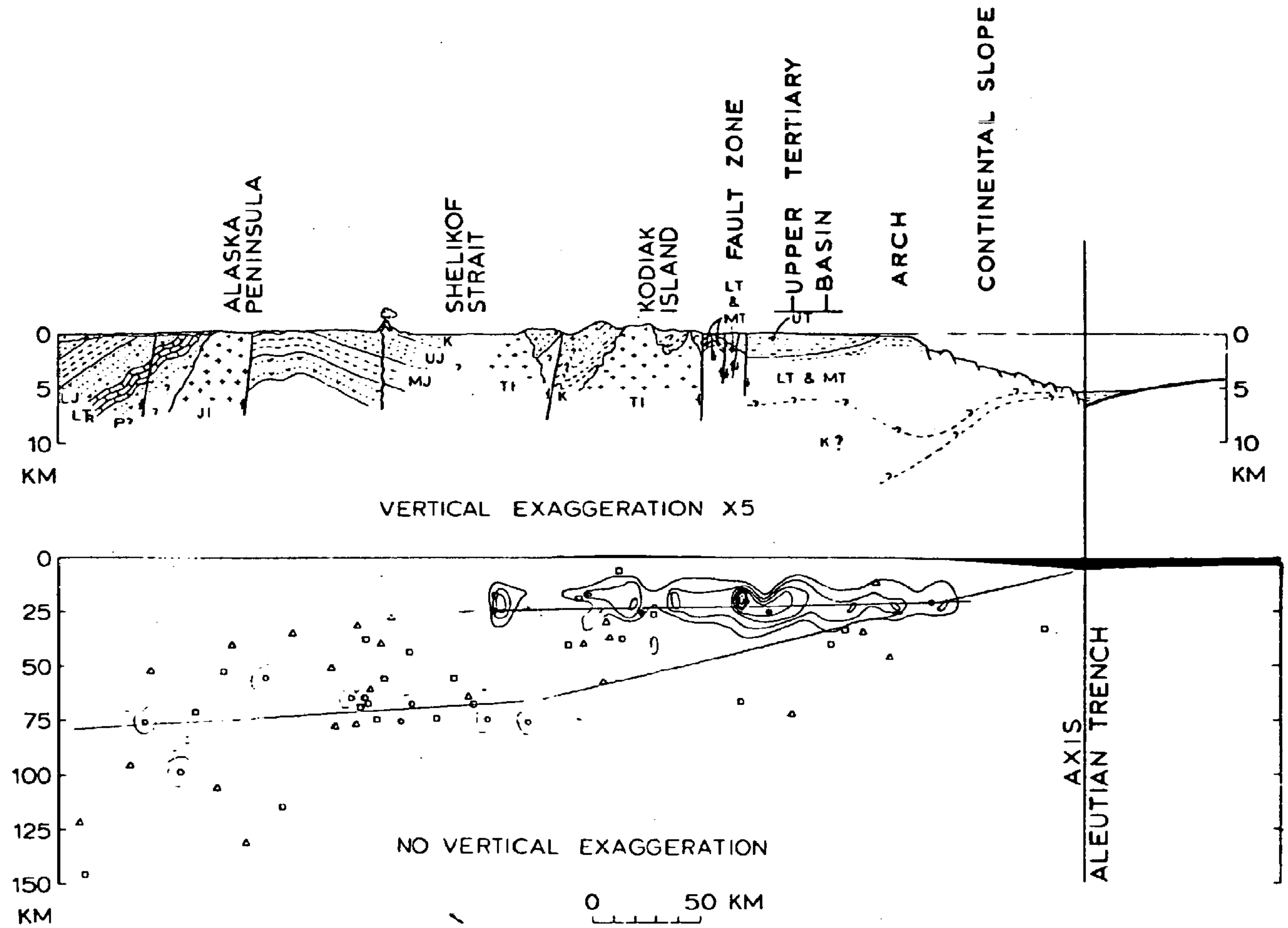


Fig. 5

FIGURE 2 Section across the Aleutian structural System showing a composite of aftershock hypocenters from the latitude of Middleton Isla the southern end of Kodiak Island. Geology generalized from Burk (1965), Moore (1969), and seismic-refraction data from Shor (1964). P, P rocks; LTR, lower Triassic rocks; LJ, MJ, and UJ, lower, middle, and upper Jurassic rocks; Ji, Jurassic intrusive rocks; K, Cretaceous rocks; L] MT, lower and middle Tertiary rocks; UT, upper Tertiary rocks; Ti, lower Tertiary intrusive rocks. Hypocenters are from Tobin and Sykes (1! where location ○ is most accurate, □ is of intermediate accuracy, and Δ is least accurate. Contours are strain release from aftershocks of the 1 Alaska earthquake in 100-unit intervals of equivalent magnitude 3.0 earthquakes per 157 km^2. Individual aftershocks greater than magnitude shown with filled circles (•). Queried contacts extrapolated from seismic-refraction stations suggest a lower and middle Tertiary thickened se possibly a continental rise, seaward of a similar Cretaceous feature.

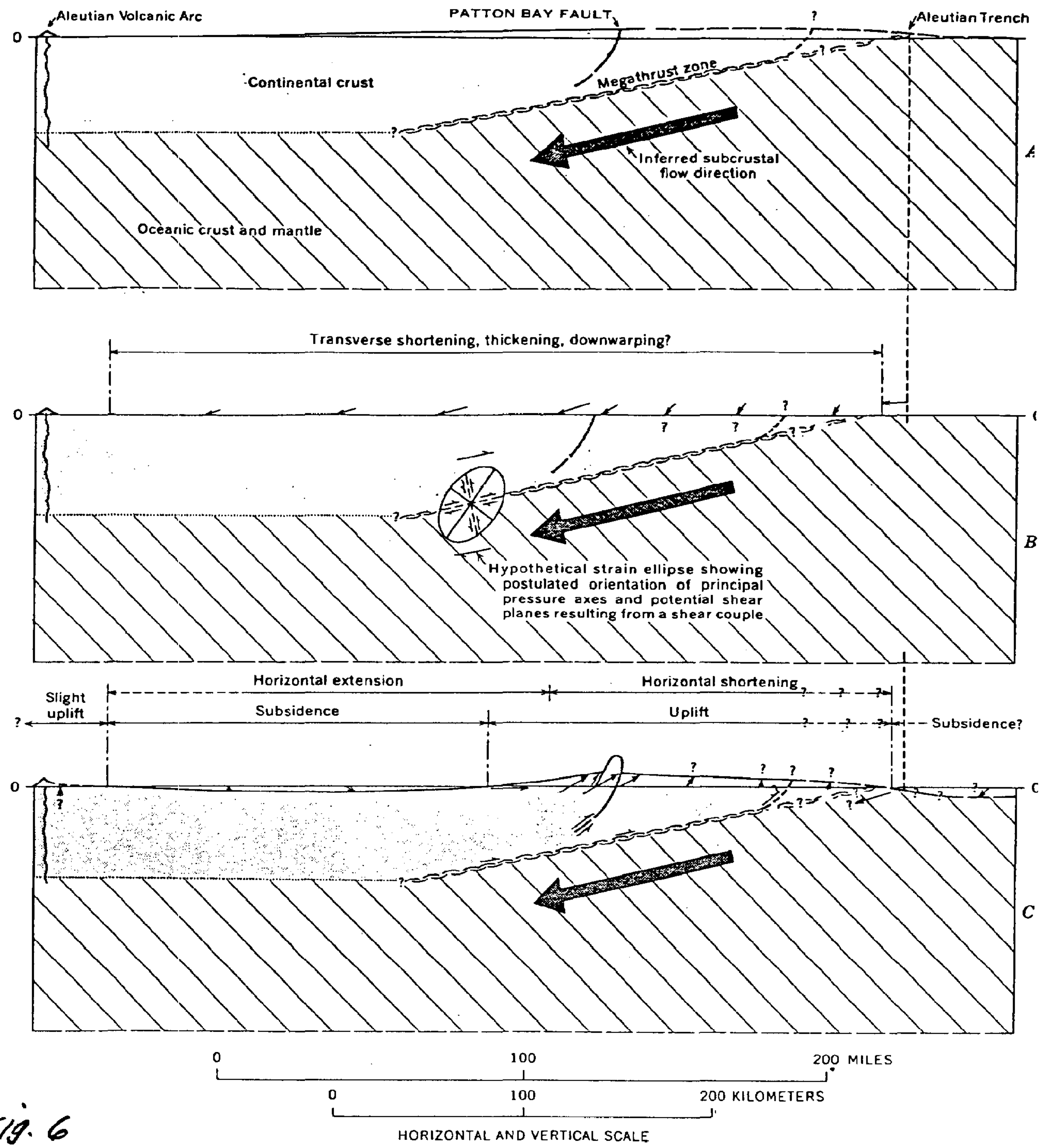
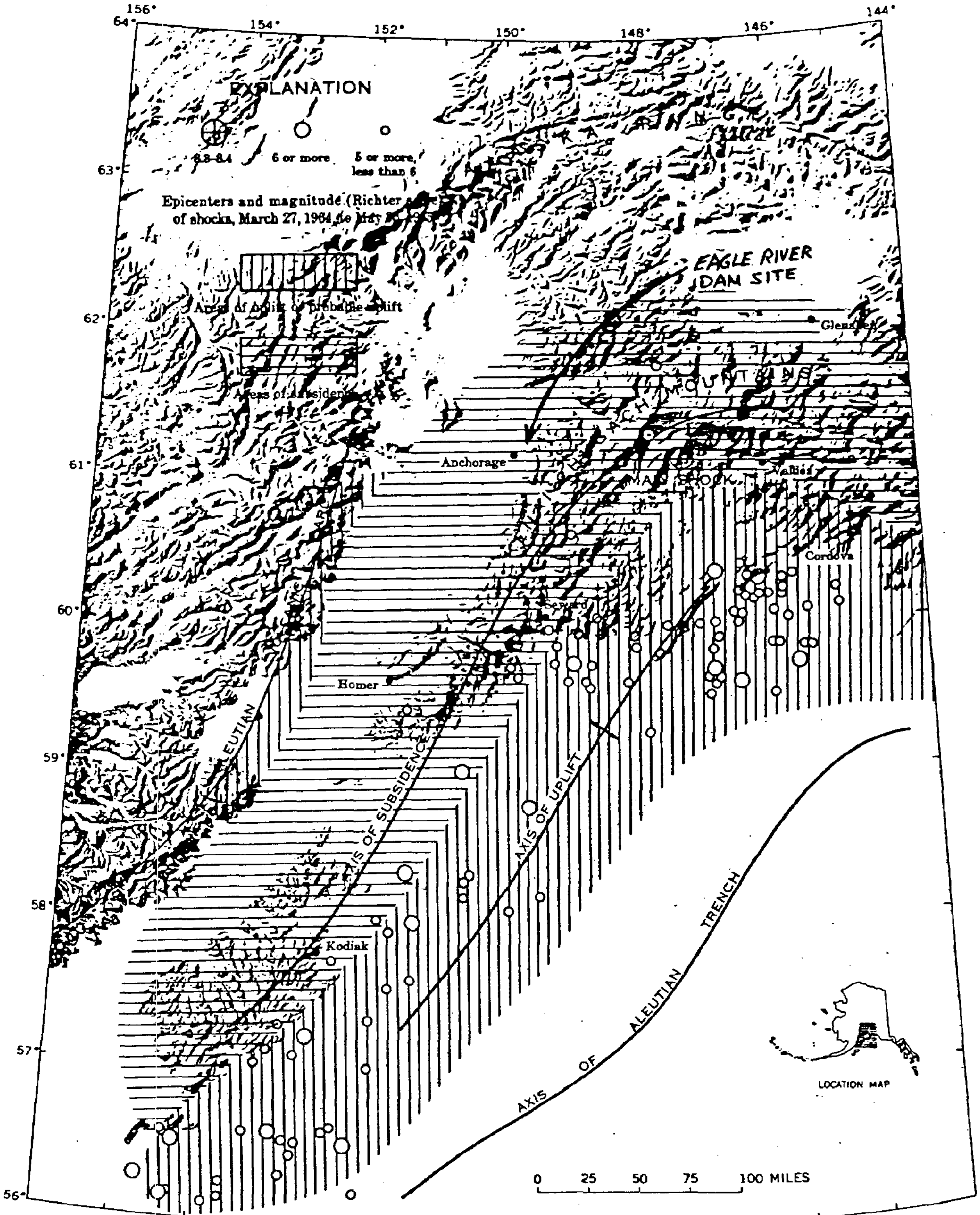


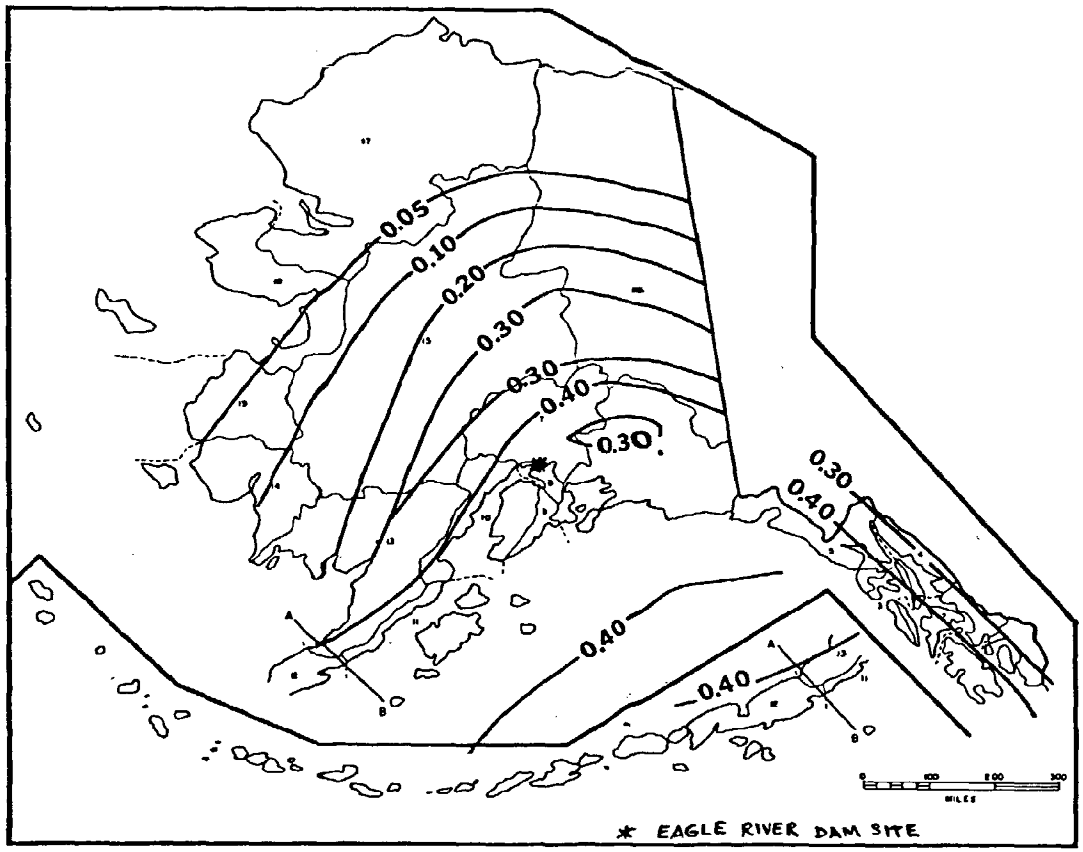
Fig. 6

42.—Diagrammatic time-sequential cross sections through the crust and upper mantle in the northern part of the region affect[ed] by the 1964 earthquake. *A*, Relatively unstrained condition after the last major earthquake. *B*, Strain buildup stage duri[ng] which the continental margin is shortened and downwarped. *C*, Observed and inferred displacements at time of the earthqua[ke] during which a segment of the continental margin is thrust seaward relative to the continent. Datum is the upper surface of t[he] crust beneath the cover of water and low-velocity sediments. Vertical displacements at the surface, which are indicated by t[he] profiles and by arrows showing sense and relative amount of movement, are about × 1,000 scale of the figure.



9.—Map of south-central Alaska, showing epicenter of March 27, 1964, earthquake, major aftershocks, and areas of tectonic land-level changes. Most aftershocks centered in the area of uplift along the continental margin of the Aleutian Trench between the Trinity Islands and the epicenter of the main shock. Data chiefly from reports by the U.S. Coast and Geodetic Survey (1964, 1965a), Grantz, Plafker, and Kachadoorian (1964), and Plafker (1965).

Fig. 7



ALASKA

Fig. 8. CONTOUR MAP FOR EFFECTIVE PEAK ACCELERATION

Exhibit D
Harding-Lawson Associates
Summary of Laboratory Tests

RF1

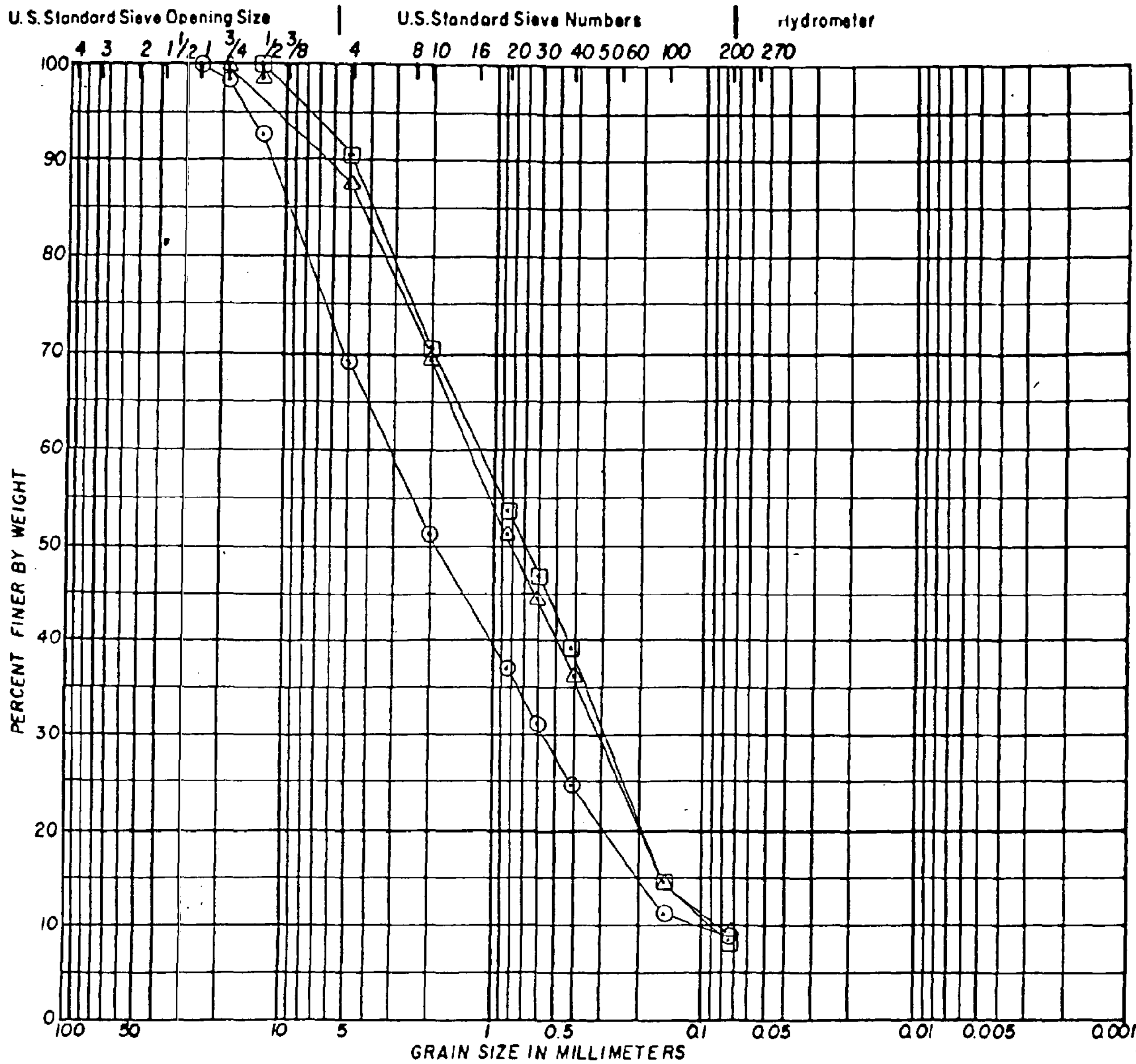
SAMPLE SOURCE	CLASSIFICATION	LAB TEST RESULTS
B-3 @ 85'	GRAY SANDY SILT (ML) with fibrous organics	Moisture Content (M)=30.3% Atterberg Limits (AL) Plate 3
B-4 @ 35,40 & 45' (combined)	BROWN SILTY GRAVELLY SAND (SP-SM)	M = 11.7% Particle Size Analysis (PSA) Plate 2
B-5 @ 5 & 10' (combined)	BROWN SILTY GRAVELLY SAND (SP-SM)	M= 14.8% PSA on Plate 2
B-5 @ 20'	GRAY SILT (ML)	M = 24.2% Minus #200 Sieve = 99.8% A.L. on Plate 3 Consolidation (consol) on Plates 4-6 Triaxial Compression Consolidated/Undrained Tx (CU) on Plate 13 Effective Stress Paths Plate 16
B-5 @ 25'	GRAY SILT (ML)	M = 26.9% A.L. on Plate 3 Unconfined Compression (UC) on Plate 17
B-5 @ 35'	DARK GRAY SILT (ML) with trace of silt	M = 28.3% Minus #200 Sieve = 96.5%
B-6 @ 5,10 & 15' (combined)	GRAY SILTY GRAVELLY SAND (SP-SM)	M = 13.7% PSA on Plate 2
B-6 @ 20'	GRAY SILT (ML)	M = 21.6% A.L. on Plate 3 Minus #200 Sieve = 99.9% Consol on Plates 7-9 Tx(CU) on Plate 14
B-6 @ 25'	GRAY SILT (ML)	M = 26.2% Minus #200 Sieve = 99.6% A.L. on Plate 3 Consol on Plates 10-12 Tx(CU) on Plate 15
B-6 @ 40'	GRAY CLAY (CL)	M = 24.8% A.L. too quick for liquid limit & non-plastic Unconfined Compression (UC) on Plate 17

ALL SAMPLING BY OTHERS

Plate 1

HARDING - LAWSON ASSOCIATES *Consulting Engineers and Geologists*	SUMMARY OF LABORATORY TESTS CH_2M-HILL EAGLE RIVER DAM PROJECT Eagle River, Alaska	PLATE 1
Job No. 5561,009.08 Appr: GJC Date 3/81		

RF 24



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Sample Source	Classification
⊙	B-4 @ 35', 40' & 45' (combined)	BROWN SILTY GRAVELLY SAND (SP-SM)
⊡	B-5 @ 5' & 10' (combined)	BROWN SILTY GRAVELLY SAND (SP-SM)
△	B-6 @ 5', 10' & 15' (combined)	GRAY SILTY GRAVELLY SAND (SP-SM)

HARDING-LAWSON ASSOCIATES

Consulting Engineers and Geologists

Job No. 5561,009.08 Appr: [illegible] Date 3/81

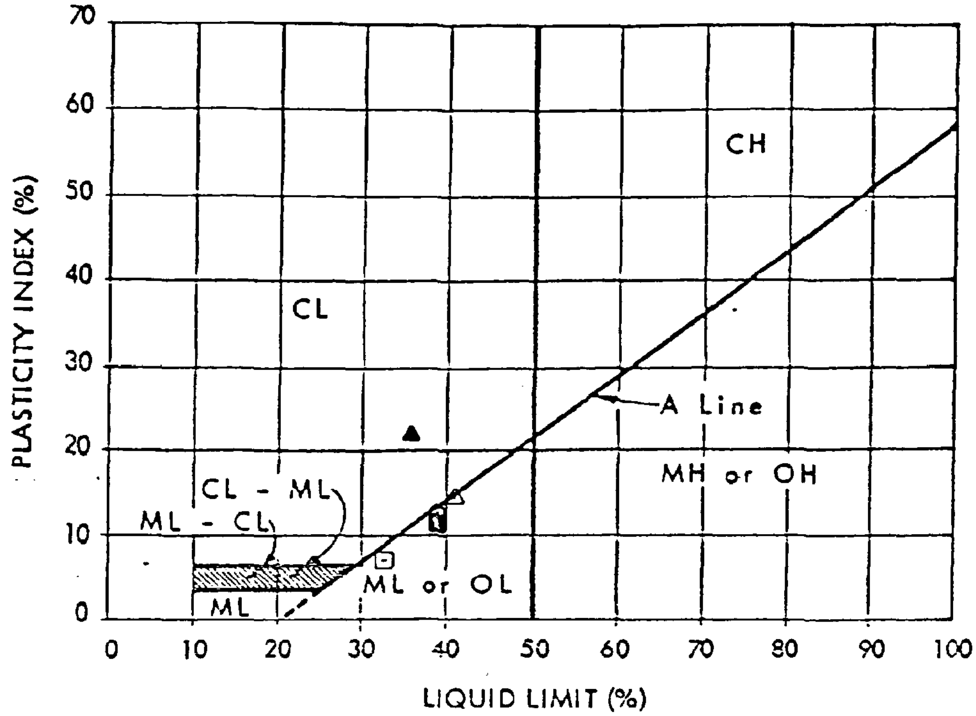
PARTICLE SIZE ANALYSIS

CH_2M-HILL
EAGLE RIVER DAM PROJECT
Eagle River, Alaska

PLATE 2

N35080

RF 25



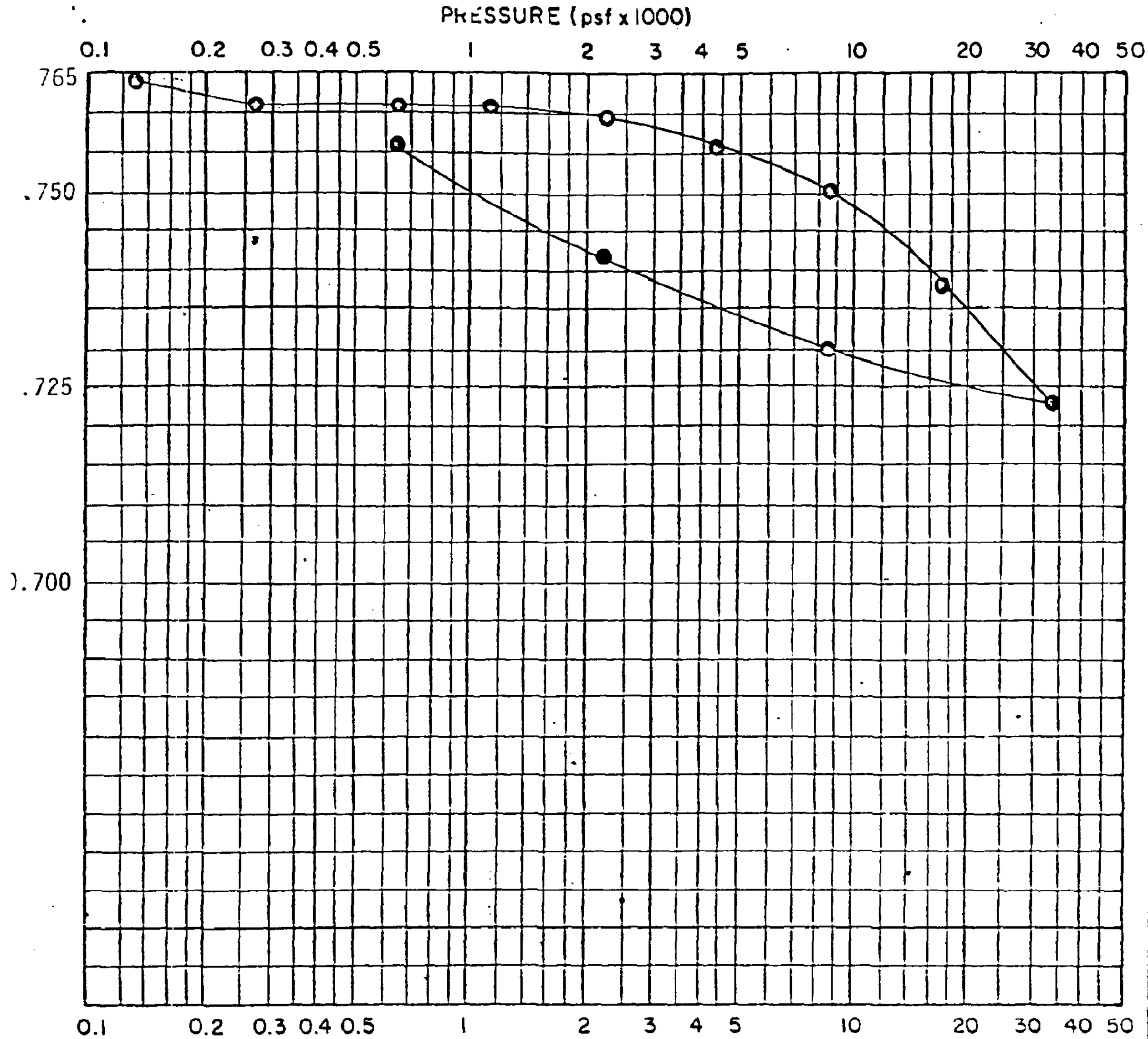
Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
⊙	GRAY SANDY SILT (ML) B-3 @ 85.0	39	26	13	
⊡	GRAY SILT (ML) B-5 @ 19.5-20.0	33	26	7	
△	GRAY SILT (ML) B-5 @ 25.0	41	27	14	
●	GRAY SILT (ML) B-6 @ 19.0-19.5	-	-	Non-plastic	99.9
■	GRAY SILT (ML) B-6 @ 24.5-25.0	39	27	12	99.6
▲	GRAY CLAY (CL) B-6 @ 40.0	36	22	14	

HARDING - LAWSON ASSOCIATES
Consulting Engineers and Geologists

Job No. 5561,009.08 Appr: [signature] Date 3/81

PLASTICITY CHART
CH_2M-HILL
EAGLE RIVER DAM PROJECT
Eagle River, Alaska

PLATE
3



TYPE OF SPECIMEN Undisturbed (trimmed)		BEFORE TEST			AFTER TEST		
DIAMETER (in.) 2.43	HEIGHT(in.) 0.80	MOISTURE CONTENT	w_o	28.4 %	w_f	28.7 %	
OVERBURDEN PRESS., P_o	- psf	VOID RATIO	e_o	0.766	e_f	0.766	
PRECONSOL. PRESS., P_c	- psf	SATURATION	S_o	100 %	S_f	100 %	
COMPRESSION INDEX, C_c	0.02	DRY DENSITY	γ_d	97 pcf	γ_d	97 pcf	
LL 33	PL 26	PI 7	G_s 2.75				
CLASSIFICATION GRAY SILT (ML)		SOURCE Boring 5 @ 19.2'					

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Consulting Engineers and Geologists.

Job No. 5561.009.08 Appr: HK Date 3/81

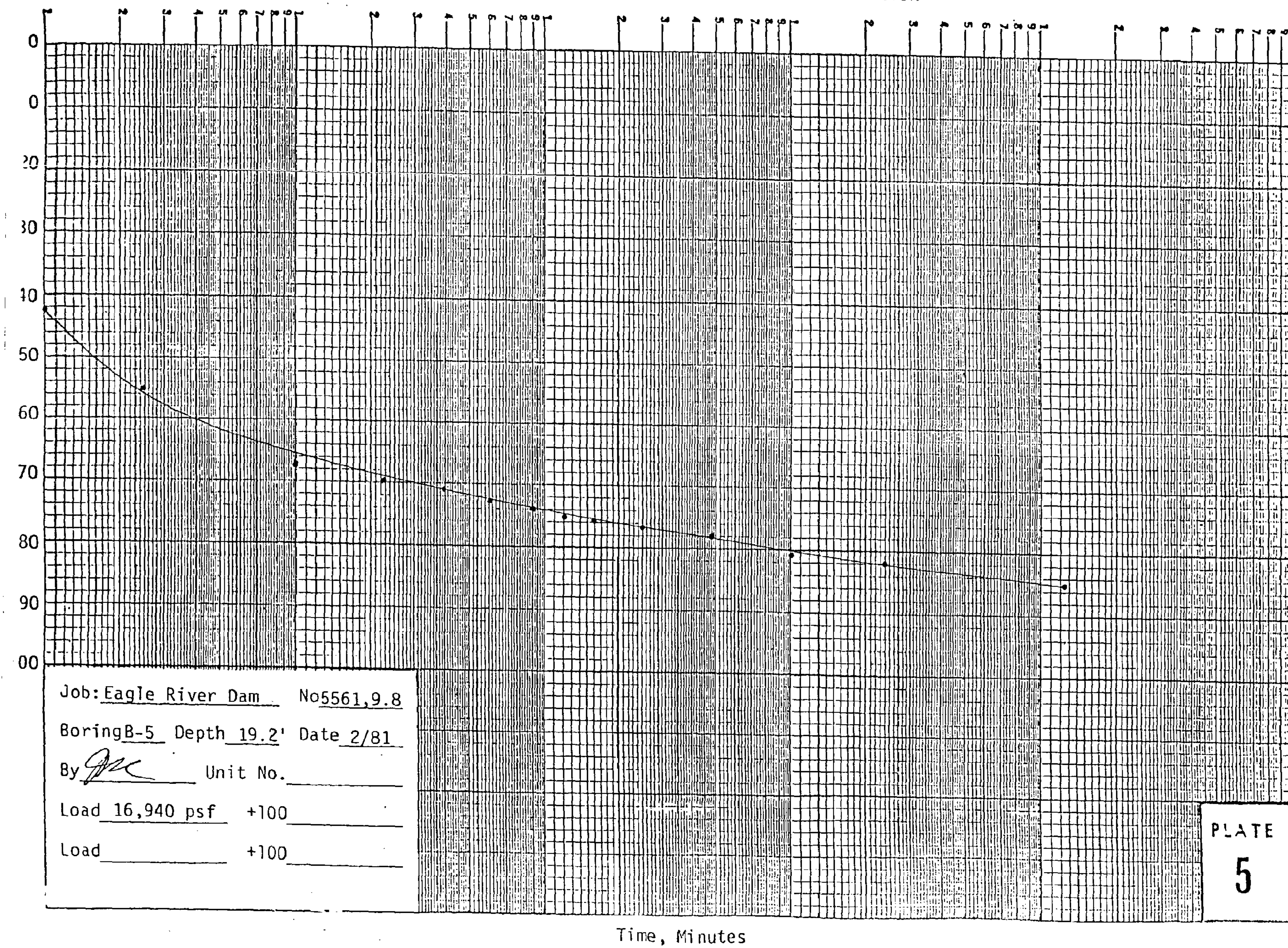
CONSOLIDATION TEST REPORT
CH_2M-HILL
EAGLE RIVER DAM PROJECT
Eagle River, Alaska

PLATE
4

7470

HARDING-LAWSON ASSOCIATES

Consolidation: Time - Deformation



Job: Eagle River Dam No 5561,9.8

Boring B-5 Depth 19.2' Date 2/81

By ____ Unit No. ____

Load 16,940 psf +100 ____

Load ____ +100 ____

PLATE 5

HARDING-LAWSON ASSOCIATES

Consolidation: Time - Deformation

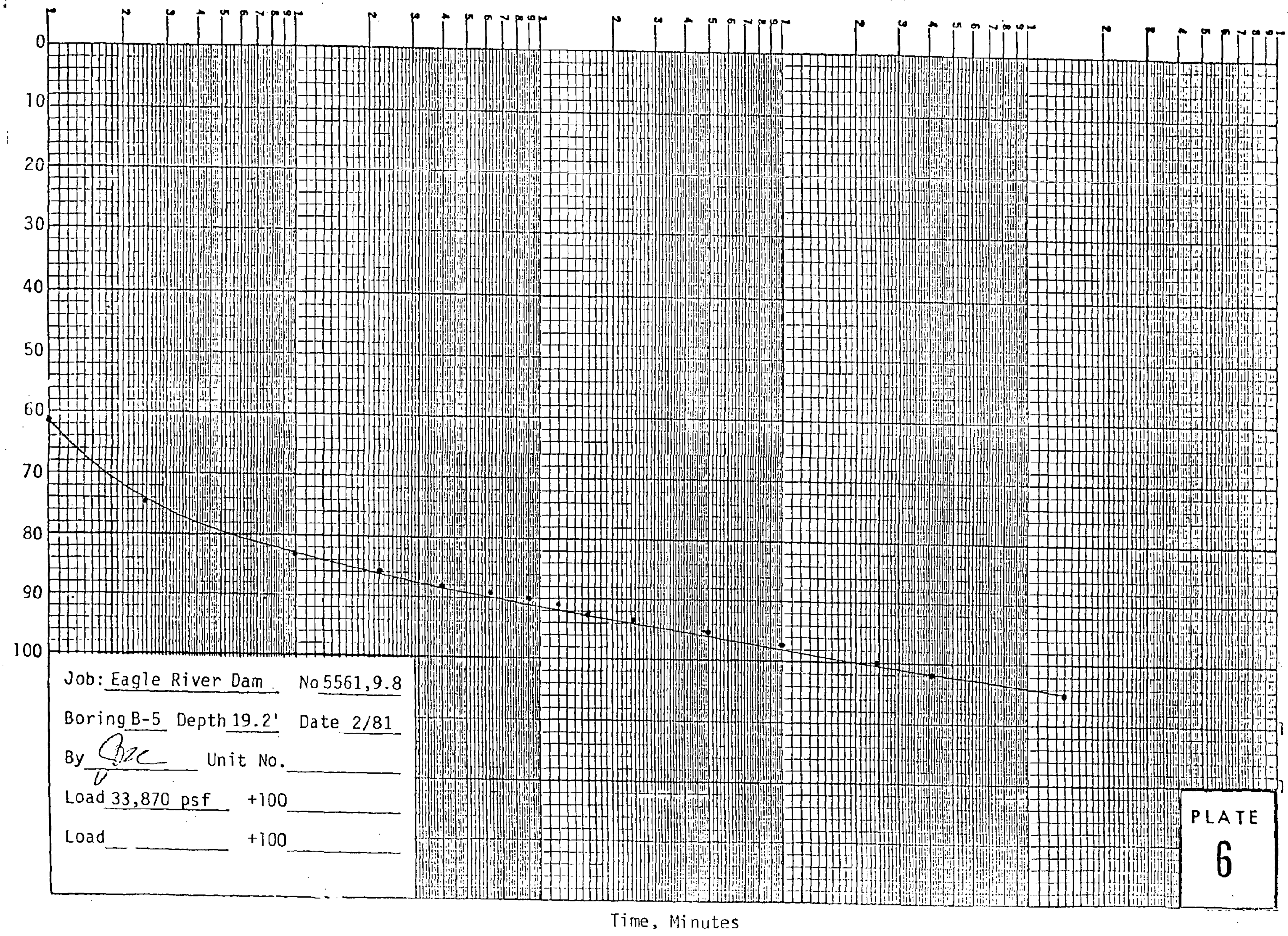
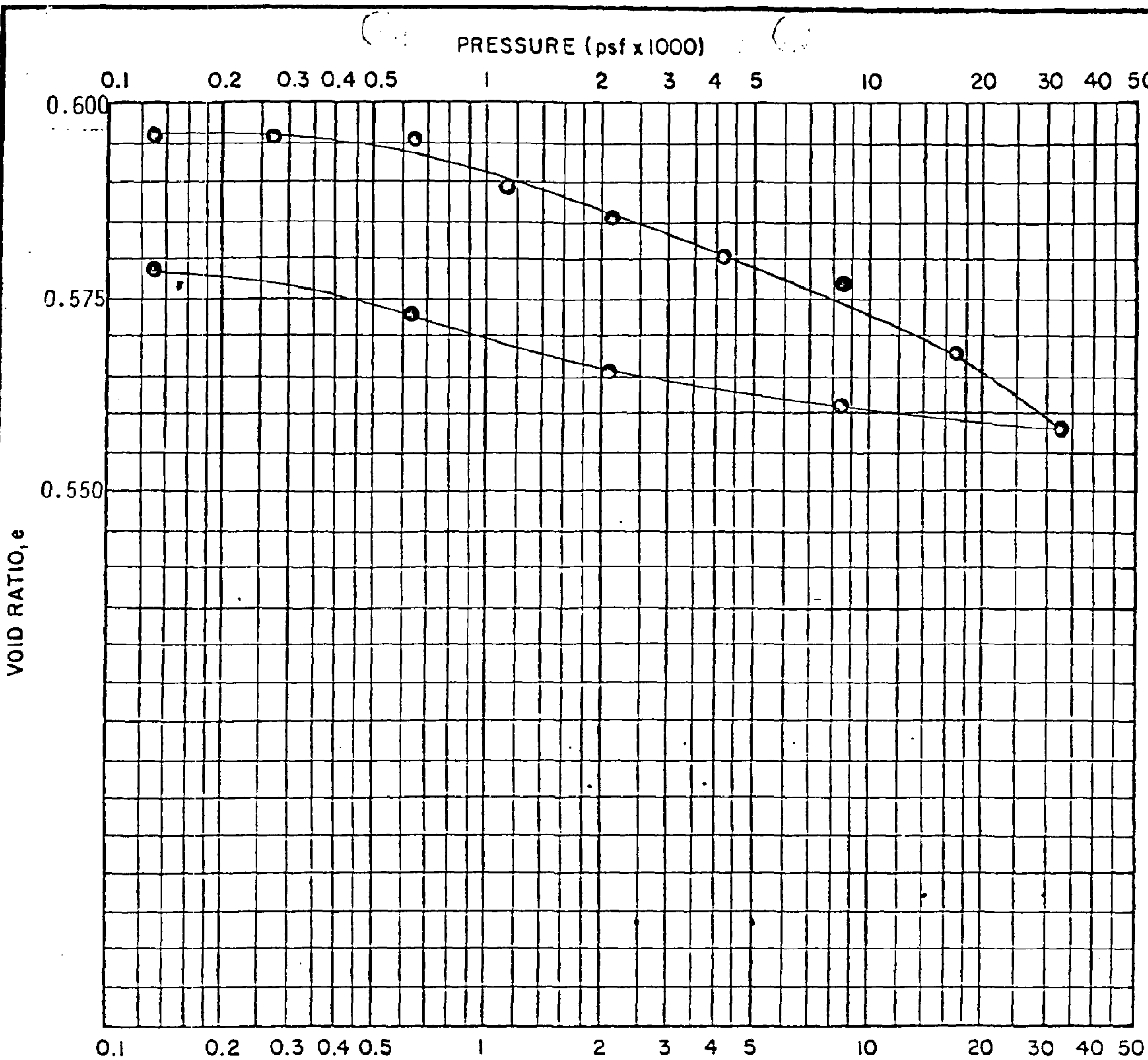


PLATE 6

RF30



TYPE OF SPECIMEN Undisturbed (trimmed)		BEFORE TEST			AFTER TEST	
DIAMETER (in.) 2.43	HEIGHT (in.) 0.80	MOISTURE CONTENT	w_o	23.5 %	w_f	23.8 %
OVERBURDEN PRESS., P_o	- psf	VOID RATIO	e_o	0.596	e_f	0.578
PRECONSOL. PRESS., P_c	- psf	SATURATION	S_o	100 %	S_f	100 %
COMPRESSION INDEX, C_c	0.01	DRY DENSITY	γ_d	106 pcf	γ_d	108 pcf
LL NV	PL NP	PI NP			G_s 2.72	
CLASSIFICATION GRAY SILT (ML)		SOURCE Boring 6 @ 19.0'				

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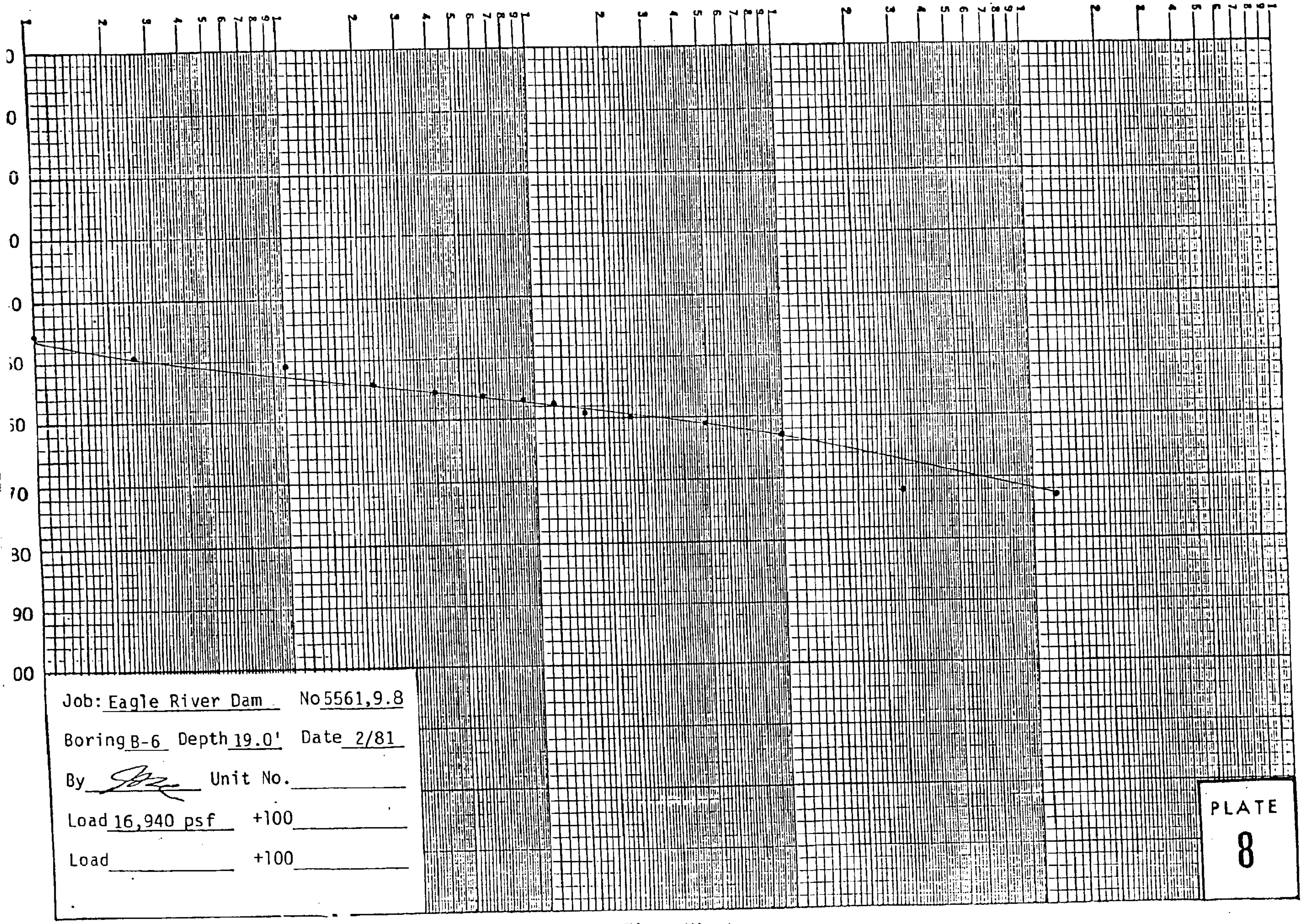
Job No. 5561,009.08 Appr [illegible] Date 3/81

CONSOLIDATION TEST REPORT
CH2M-HILL
EAGLE RIVER DAM PROJECT
Eagle River, Alaska

PLATE
7

HARDING-LAWSON ASSOCIATES

Consolidation: Time - Deformation



Job: Eagle River Dam No 5561,9.8

Boring B-6 Depth 19.0' Date 2/81

By [signature] Unit No. ______

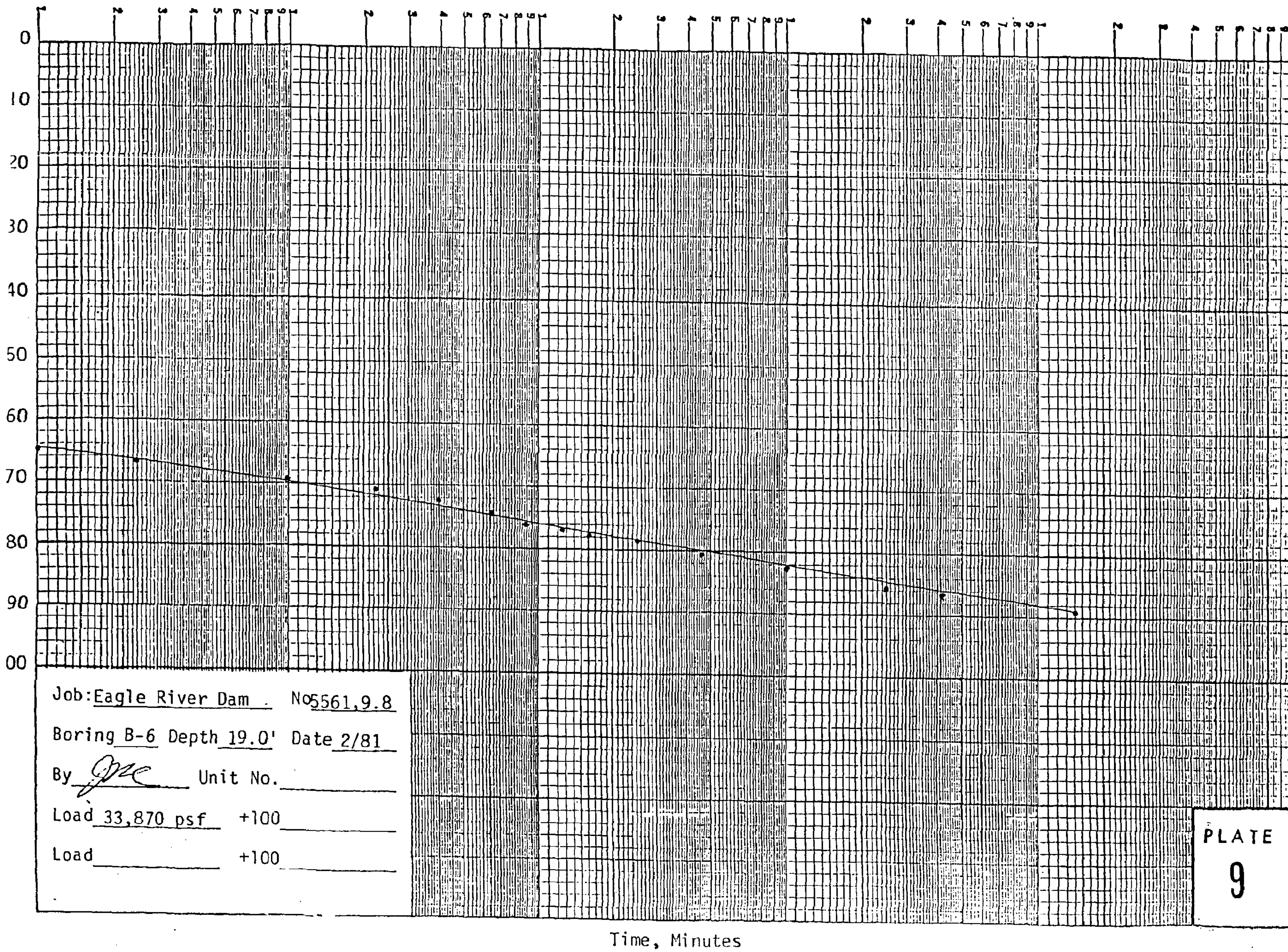
Load 16,940 psf +100 ______

Load ______ +100 ______

PLATE 8

HARDING-LAWSON ASSOCIATES

Consolidation: Time - Deformation



Job: Eagle River Dam No 5561,9.8

Boring B-6 Depth 19.0' Date 2/81

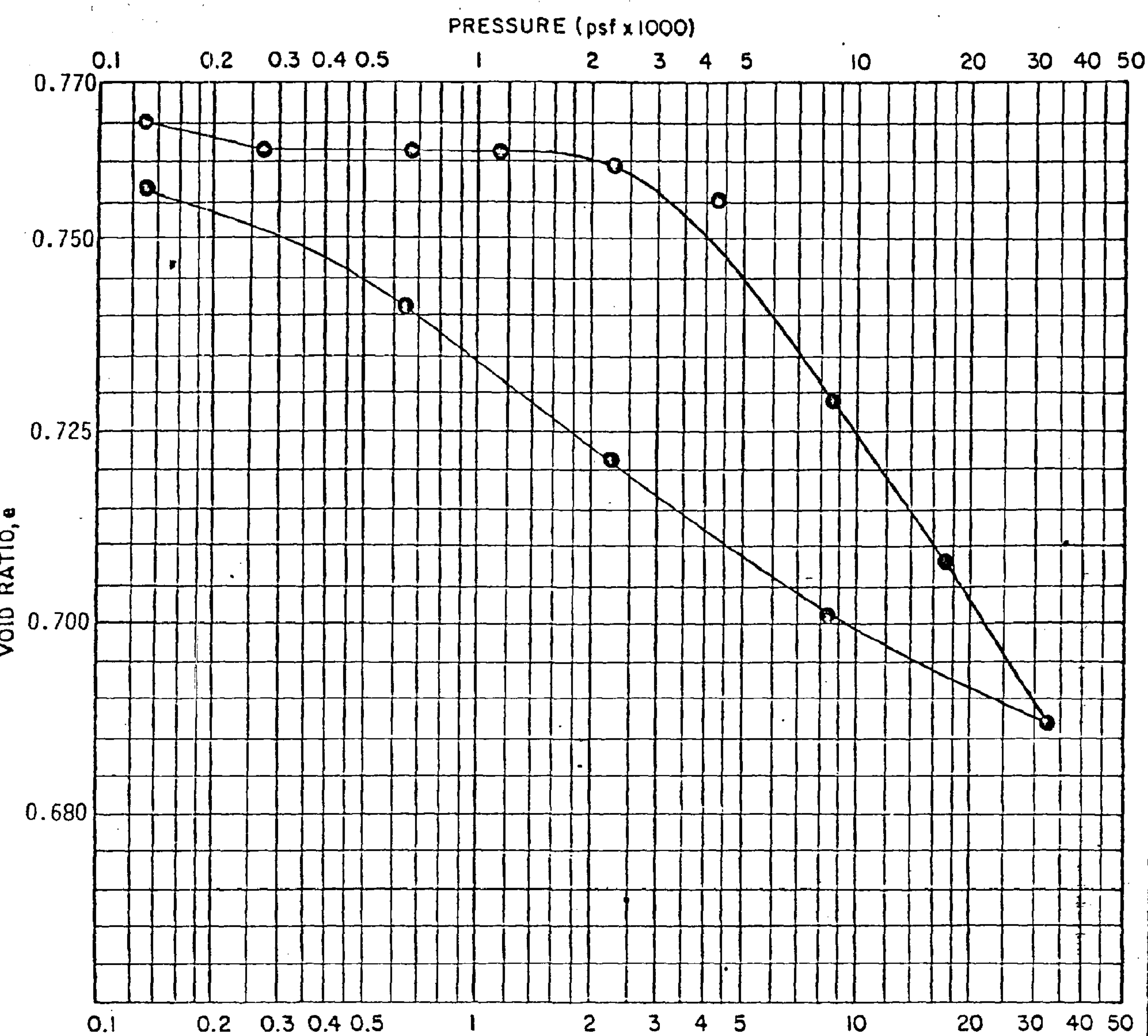
By [signature] Unit No.

Load 33,870 psf +100

Load +100

PLATE 9

RF30



TYPE OF SPECIMEN Undisturbed (trimmed)			BEFORE TEST				AFTER TEST	
DIAMETER (in.) 2.43	HEIGHT(in.) 0.80		MOISTURE CONTENT	w_o	28.9 %		w_f	29.2 %
OVERBURDEN PRESS., P_o	-	psf	VOID RATIO	e_o	0.770		e_f	0.758
PRECONSOL. PRESS., P_c	-	psf	SATURATION	S_o	100 %		S_f	100 %
COMPRESSION INDEX, C_c	0.03		DRY DENSITY	γ_d	97 pcf		γ_d	97 pcf
LL 39	PL 27		PI 12		Gs 2.74			
CLASSIFICATION GRAY SILT (ML)			SOURCE Boring 6 @ 24.3'					

HARDING - LAWSON ASSOCIATES
Consulting Engineers and Geologists

Job No. 5561,004.08 Appr: [illegible] Date 3/81

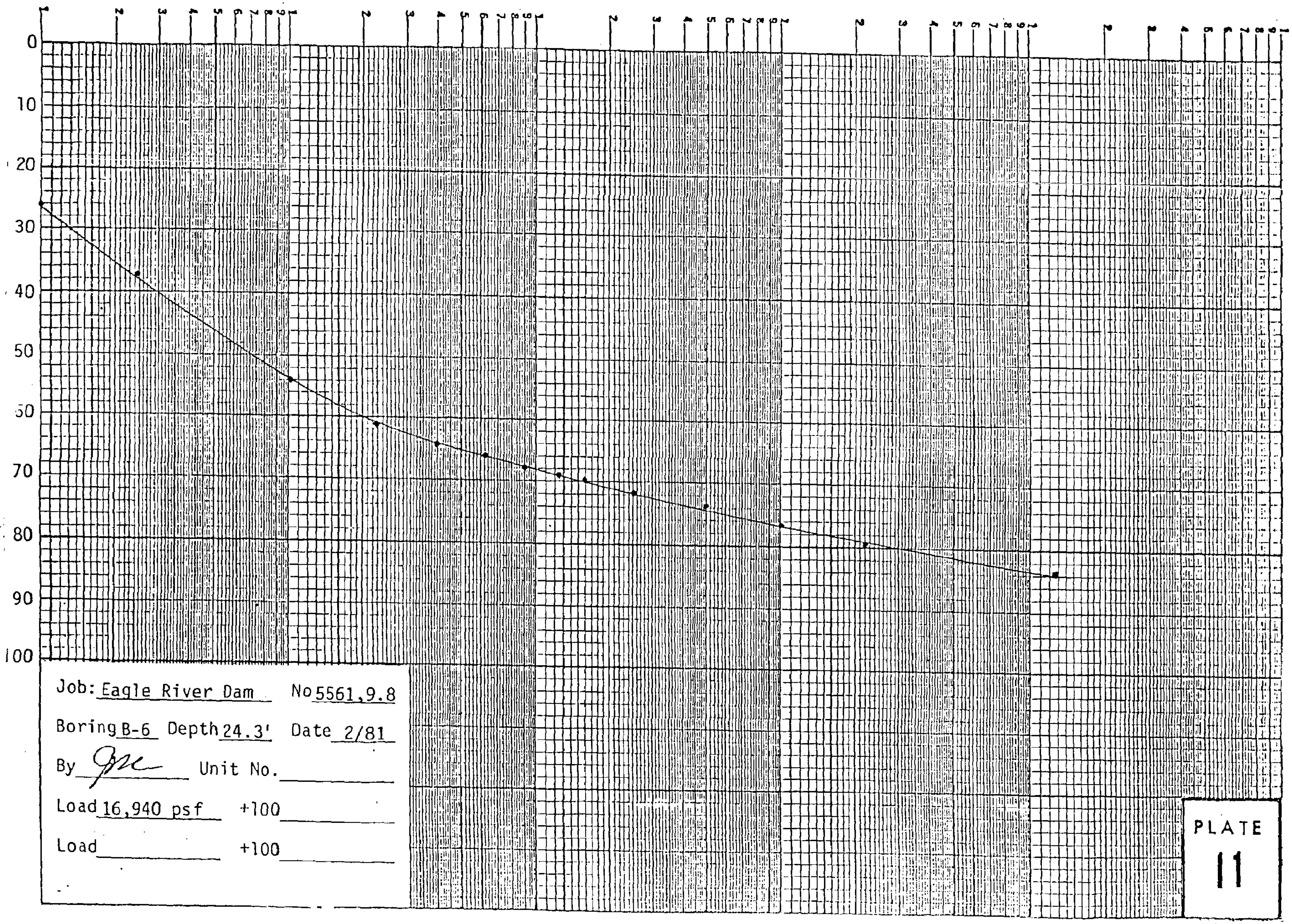
CONSOLIDATION TEST REPORT
CH2M-HILL
EAGLE RIVER DAM PROJECT
Eagle River, Alaska

PLATE 10

H02470

HARDING-LAWSON ASSOCIATES

Consolidation: Time - Deformation



Job: Eagle River Dam No 5561,9.8

Boring B-6 Depth 24.3' Date 2/81

By [signature] Unit No.

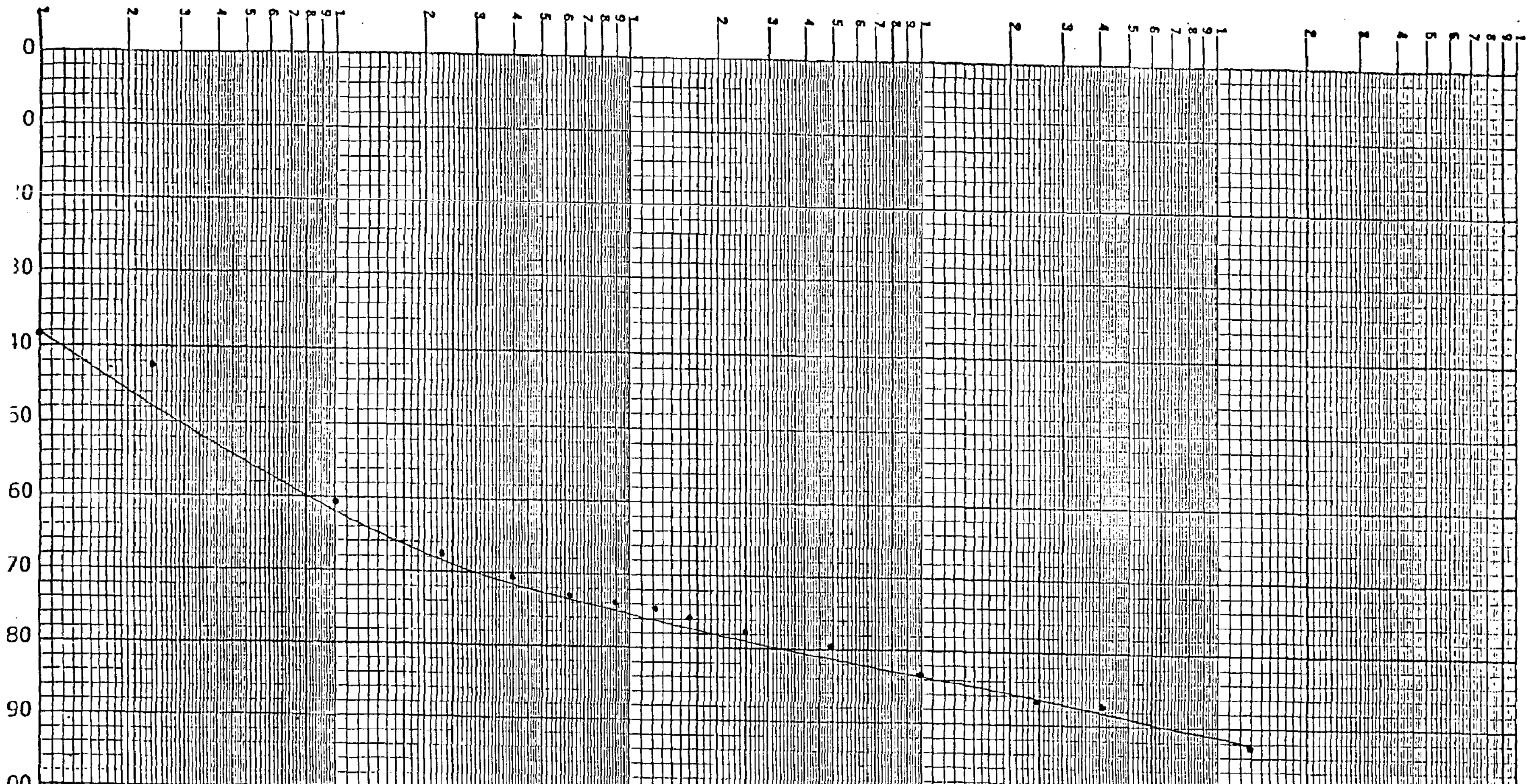
Load 16,940 psf +100

Load +100

PLATE 11

HARDING-LAWSON ASSOCIATES

Consolidation: Time - Deformation



Job: Eagle River Dam No 5561,9.8

Boring B-6 Depth 24.3' Date 2/81

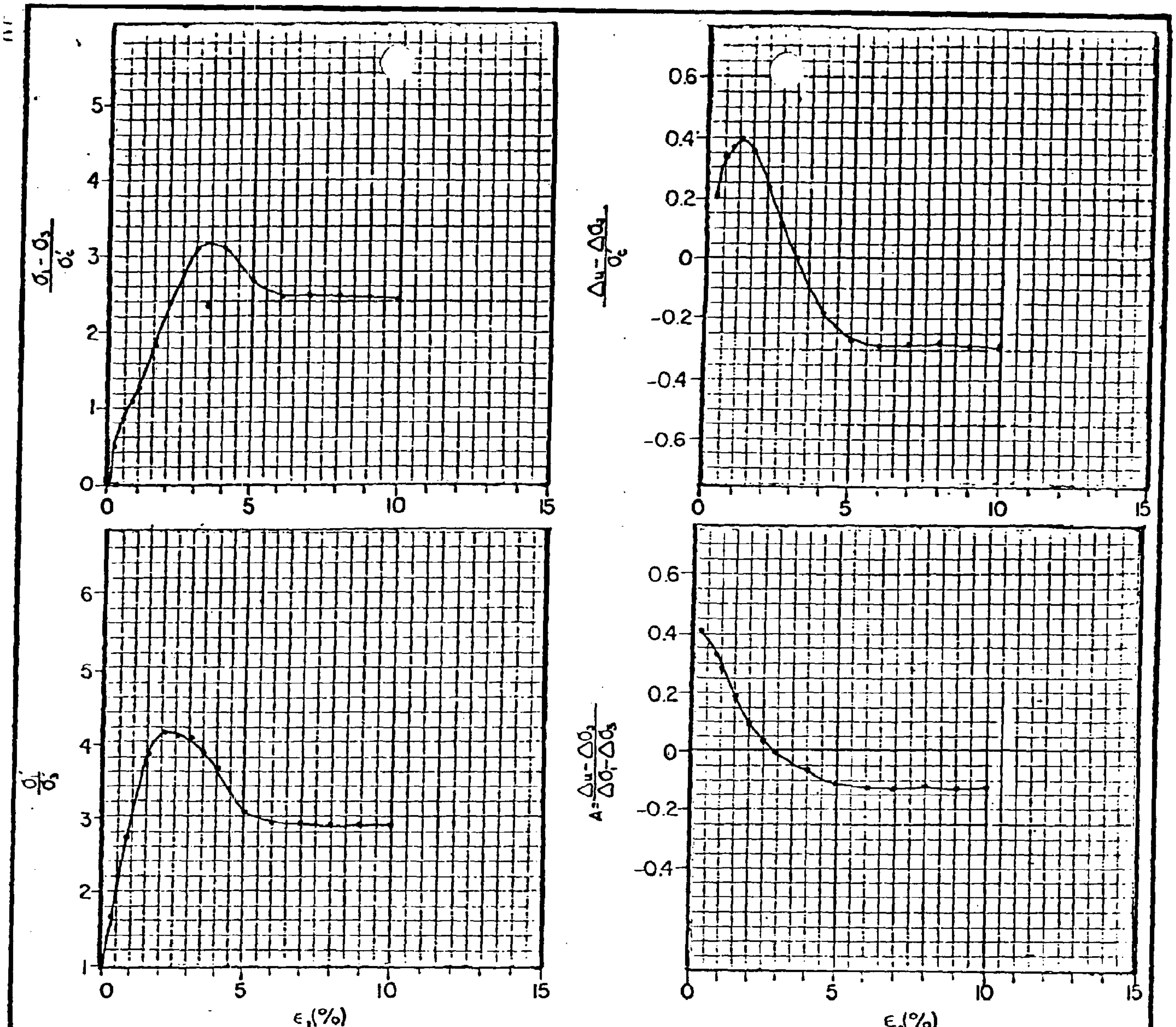
By JJK Unit No. ______

Load 33,870 psf +100 ______

Load ______ +100 ______

Time, Minutes

PLATE 12



TYPE OF SPECIMEN Undisturbed (trimmed)				BEFORE TEST			AFTER TEST	
DIAMETER (in.) 2.43	HEIGHT (in.) 6.00		MOISTURE CONTENT	w_o	25.9 %	w_f	26.8 %	
OVERBURDEN PRESS., P_0	-	psf	VOID RATIO	e_o	0.696	e_f	0.645	
CONSOLIDATION PRESS., σ_3	4320	psf	SATURATION	S_o	100 %	S_f	100 %	
STRAIN RATE	0.05	%/min	DRY DENSITY	γ_d	101 pcf	γ_d	104 pcf	
LL 33	PL 26		PI 7		G_s 2.75			
CLASSIFICATION GRAY SILT (ML)			SOURCE Boring 5 @ 19.5'					

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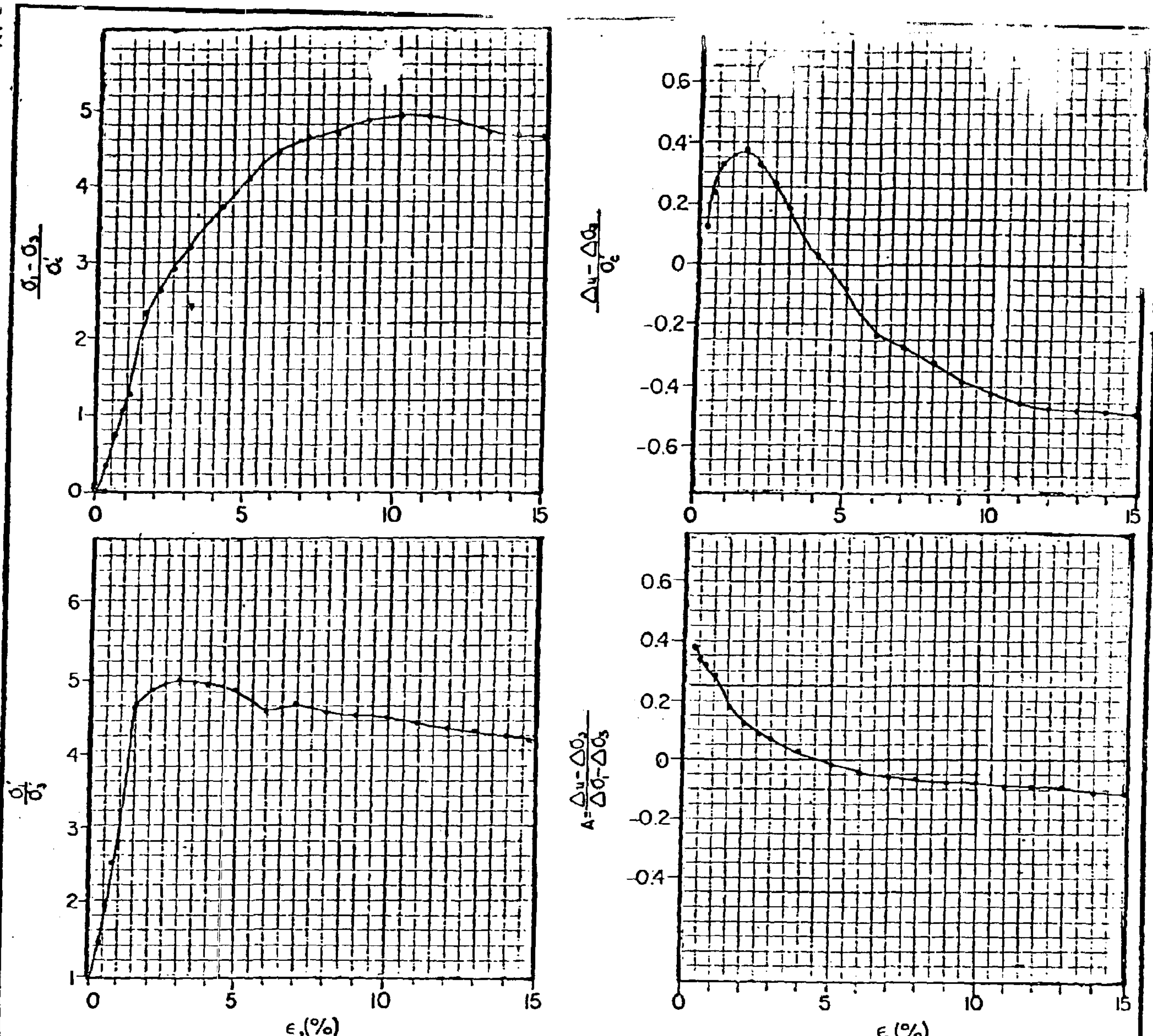
Job No. 5561.009.08 Appr. [signature] Date 3/81

CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST REPORT
CH2M-HILL
EAGLE RIVER DAM PROJECT
Eagle River, Alaska

PLATE 13



TYPE OF SPECIMEN Undisturbed (trimmed)				BEFORE TEST			AFTER TEST	
DIAMETER (in.) 2.43	HEIGHT (in.) 6.00		MOISTURE CONTENT	w_o	25.6 %	w_f	25.3 %	
OVERBURDEN PRESS., P_o	-	psf	VOID RATIO	e_o	0.676	e_f	0.613	
CONSOLIDATION PRESS., σ_3	8640	psf	SATURATION	S_o	100 %	S_f	100 %	
STRAIN RATE	0.05	%/min	DRY DENSITY	γ_d	101 pcf	γ_d	105 pcf	

LL NV	PL NP	PI NP	G_s 2.72
CLASSIFICATION GRAY SILT (ML)		SOURCE Boring 6 @ 19.5'	

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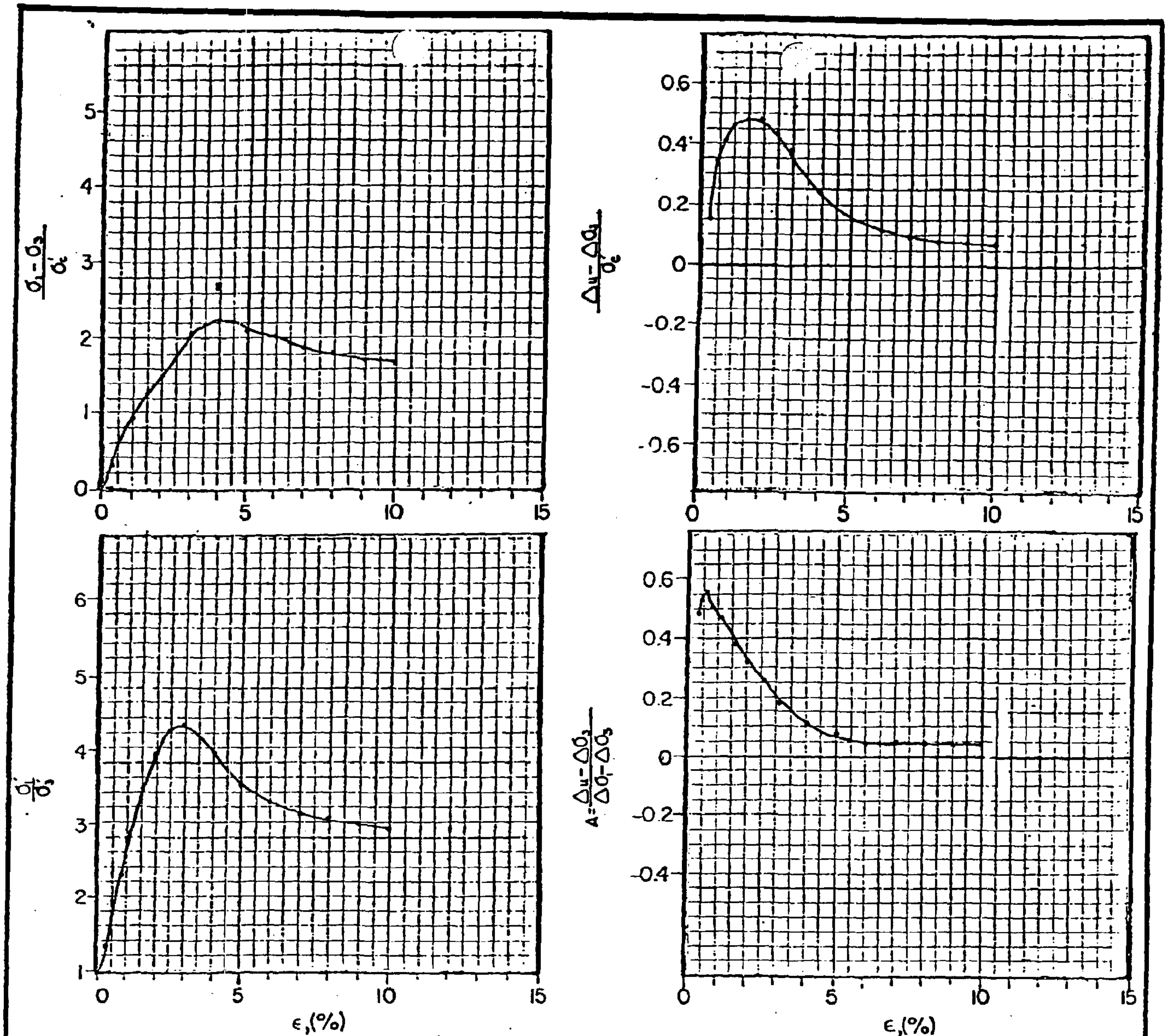
Job No. 5561,009.08 Appr. [signature] Date 3/81

CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST REPORT
CH_2M-HILL
EAGLE RIVER DAM PROJECT
Eagle River, Alaska

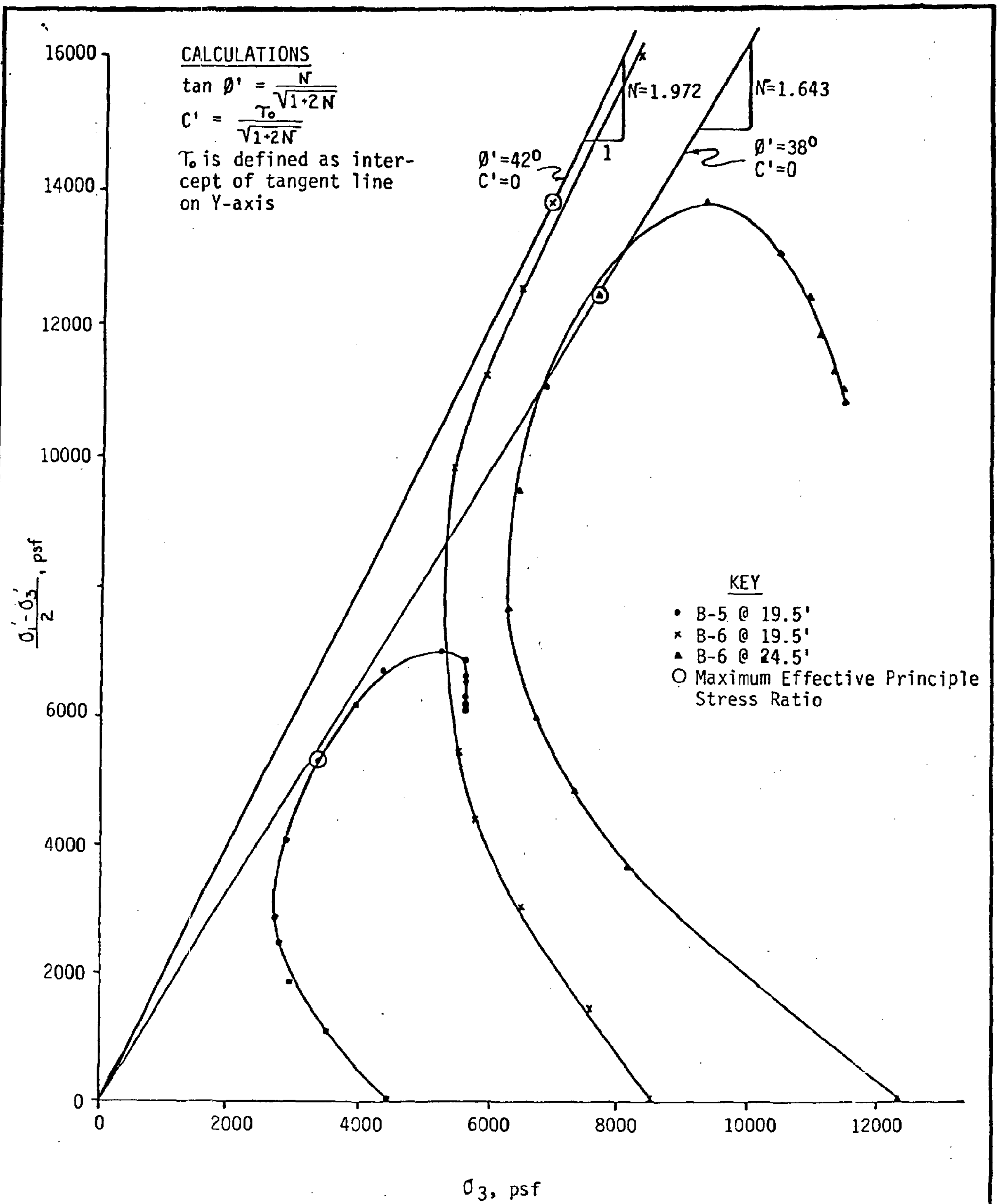
PLATE 14



TYPE OF SPECIMEN Undisturbed (trimmed)			BEFORE TEST			AFTER TEST	
DIAMETER (in.) 2.43	HEIGHT (in.) 5.82		MOISTURE CONTENT	w_o	26.6 %	w_f	26.6 %
OVERBURDEN PRESS., P_o	–	psf	VOID RATIO	e_o	0.676	e_f	0.556
CONSOLIDATION PRESS., σ_3	12240	psf	SATURATION	S_o	100 %	S_f	100 %
STRAIN RATE	0.05	%/min	DRY DENSITY	γ_d	102 pcf	γ_d	110 pcf
LL 39	PL 27		PI 12		G_s 2.74		
CLASSIFICATION GRAY SILT (ML)			SOURCE Boring 6 @ 24.5				

HARDING-LAWSON ASSOCIATES *Consulting Engineers and Geologists*	CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST REPORT CH2M-HILL EAGLE RIVER DAM PROJECT Eagle River, Alaska	PLATE 15
Job No. 5561,009.08 Appr. [signature] Date 3/81		

HL 2473

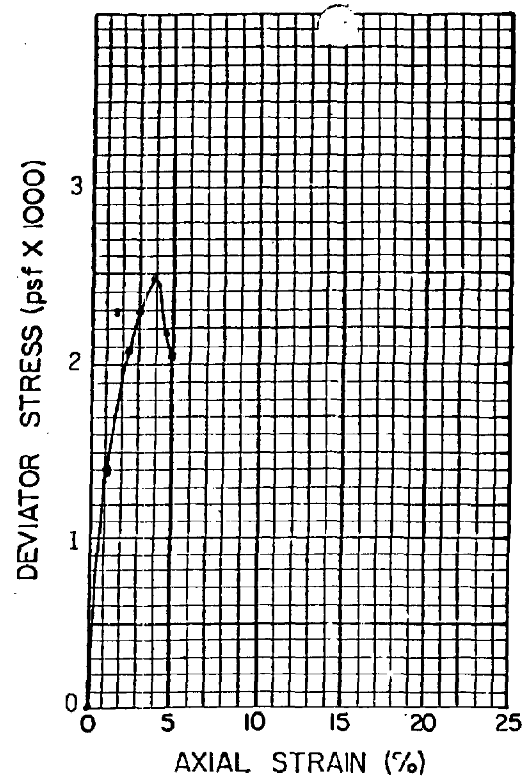


HARDING-LAWSON ASSOCIATES
Consulting Engineers and Geologists

Job No. 5561,009.08 Appr: [signature] Date 3/81

EFFECTIVE STRESS PATHS
CH2M-HILL
EAGLE RIVER DAM PROJECT
Eagle River, Alaska

PLATE
16

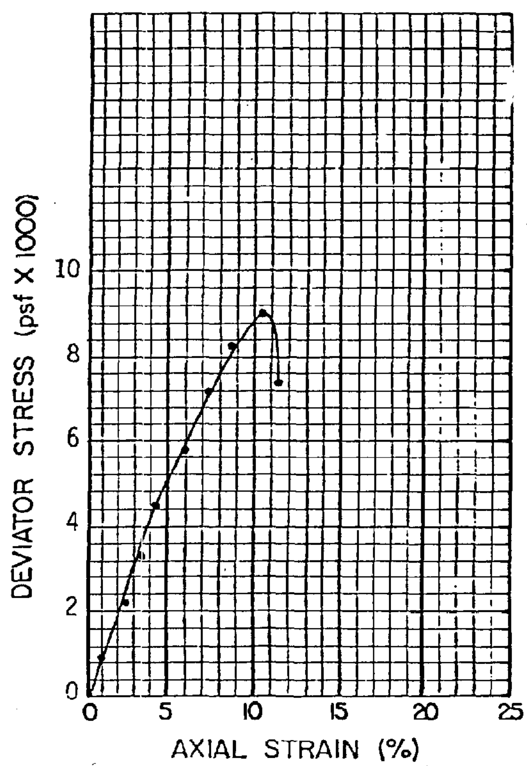


DIAMETER (in): 1.40
HEIGHT (in): 3.00

MOISTURE CONTENT (%): 26.9
DRY DENSITY (pcf): 104

CELL PRESSURE (psf): NONE
SHEAR STRENGTH (psf): 1230

SAMPLE SOURCE: Boring 5 @ 25.0'
DESCRIPTION: GRAY SILT (ML)



DIAMETER (in): 1.40
HEIGHT (in): 3.00

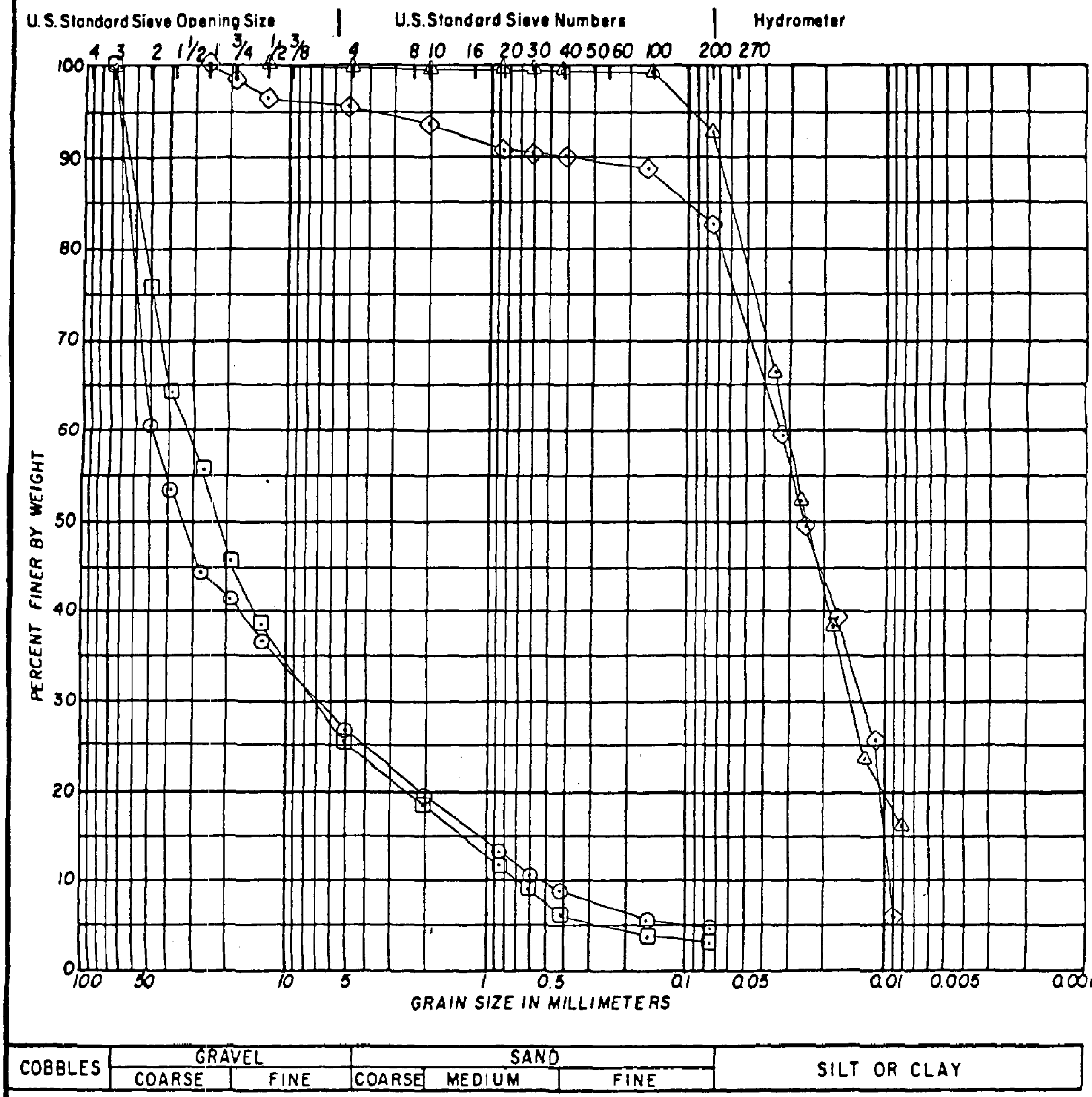
MOISTURE CONTENT (%): 24.8
DRY DENSITY (pcf): 112

CELL PRESSURE (psf): NONE
SHEAR STRENGTH (psf): 4530

SAMPLE SOURCE: Boring 6 @ 40.0'
DESCRIPTION: GRAY SILT (ML)

HARDING - LAWSON ASSOCIATES *Consulting Engineers and Geologists*	TRIAXIAL COMPRESSION TEST REPORT UNCONSOLIDATED - UNDRAINED	PLATE
Job No. 5561,009.08 Appr: [illegible] Date 3/81	CH2M-HILL EAGLE RIVER DAM PROJECT Eagle River, Alaska	17

RF24



Symbol	Sample Source	Classification
⊙	Bulk Composite Sample (Bulk Bags 1W)	BROWN SANDY GRAVEL (GW)
⊡	Bulk Composite Sample (A S&G 1E)	BROWN SANDY GRAVEL (GW)
△	B-1 @ 5'+ and 10'+(2 jars)	GRAY SANDY SILT (ML)
◇	B-2 @ 5'+ (1 jar)	GRAY SANDY SILT (ML)

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Job No. 5561,009.08 Appr: [signature] Date 2/19/81

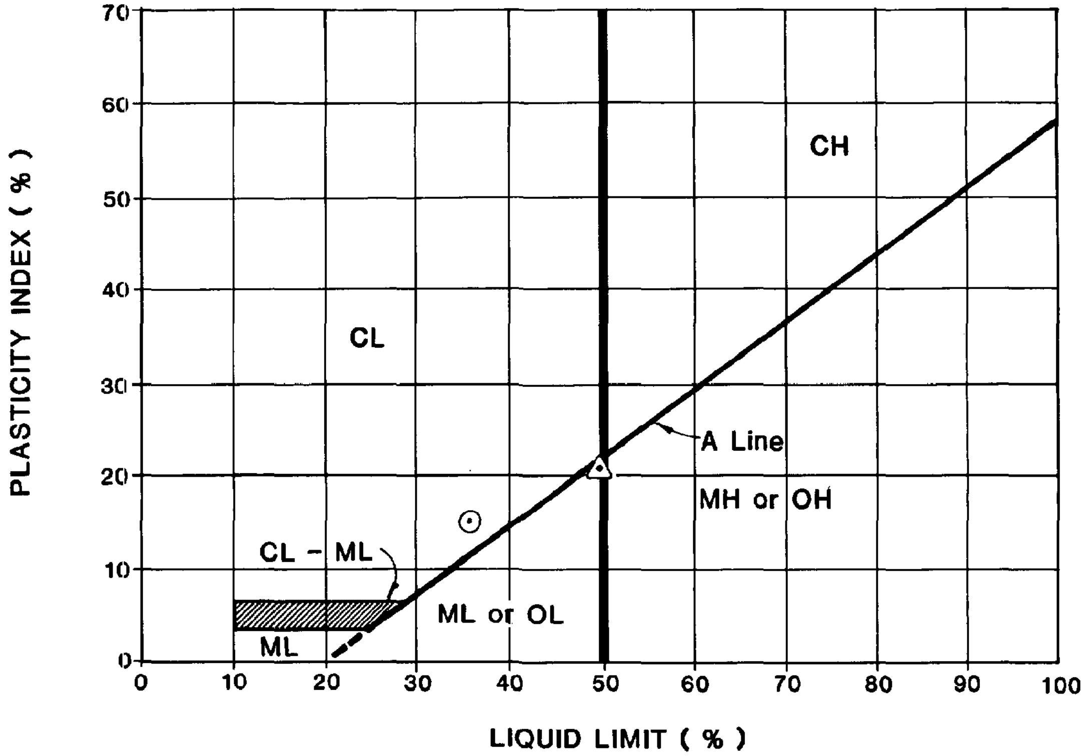
PARTICLE SIZE ANALYSIS

EAGLE RIVER DAM PROJECT
Eagle River, Alaska

PLATE 18

N35080

Exhibit E
CH2M HILL
Summary of Laboratory Tests

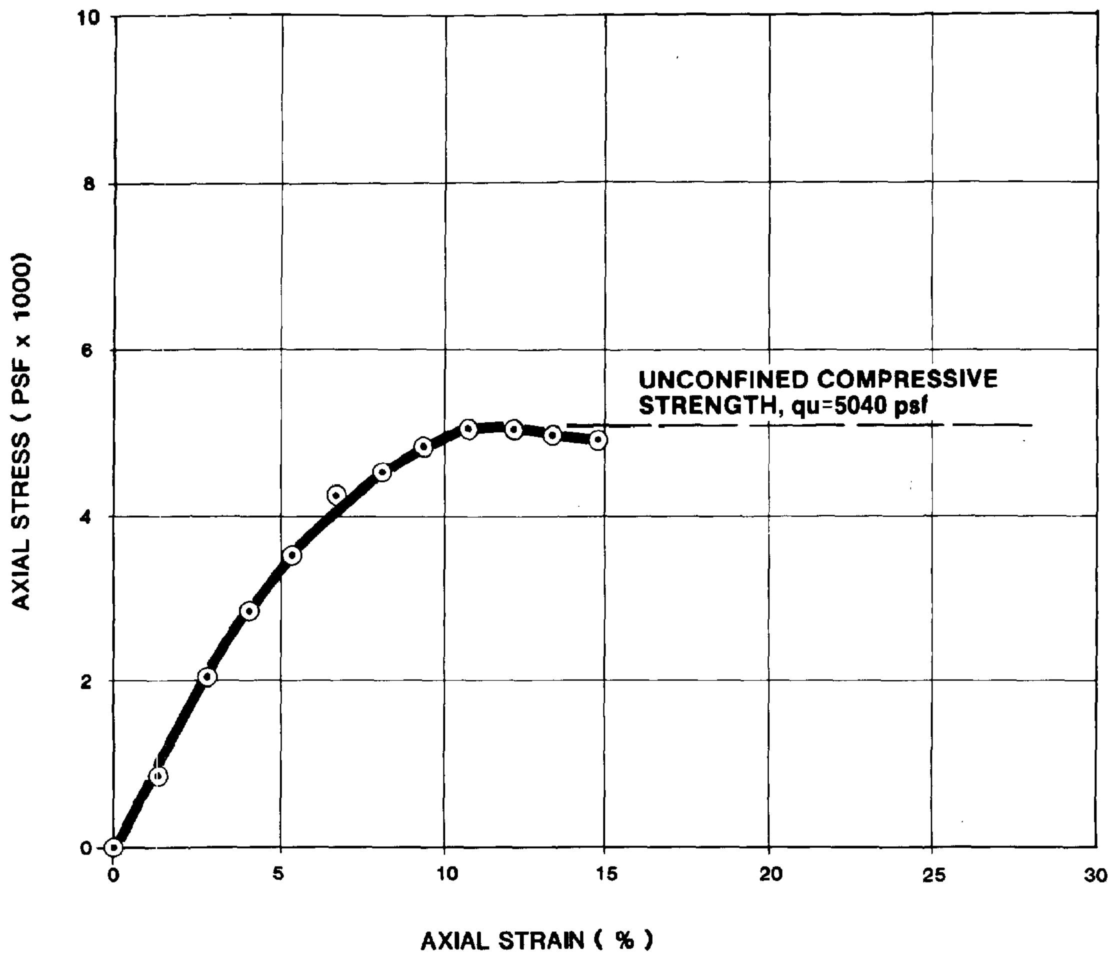


SAMPLE DATA:

⊙ GRAY CLAY (CL)
B-5 SS-8
DEPTH = 45 ft.
LIQUID LIMIT = 36 %
PLASTIC LIMIT = 21 %
PLASTICITY INDEX = 15
NATURAL MOISTURE CONTENT = 22%

△ GRAY SILT (ML)
B-6 SS-4
DEPTH = 30 ft.
LIQUID LIMIT = 49 %
PLASTIC LIMIT = 28 %
PLASTICITY INDEX = 21
NATURAL MOISTURE CONTENT = 29%

Figure E-1
Plasticity Chart
Eagle River Dam Project

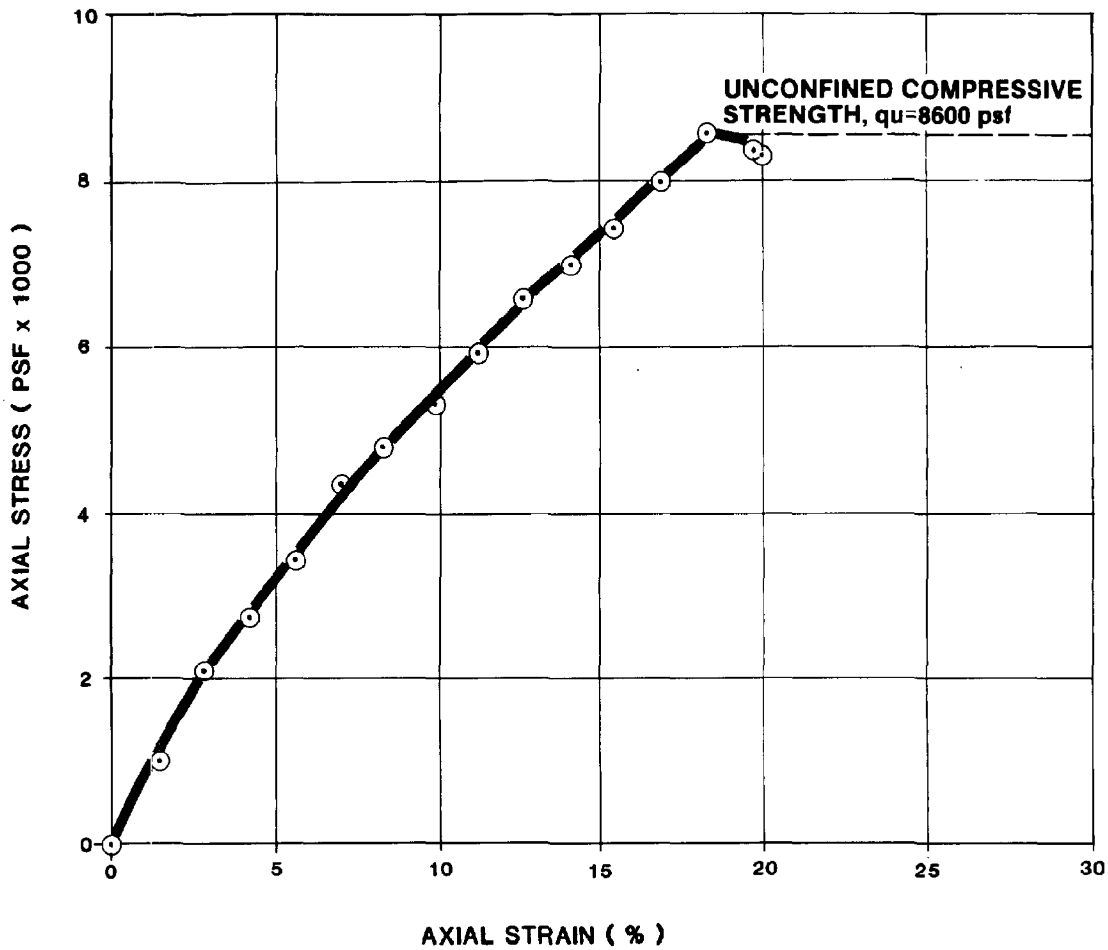


RATE OF STRAIN = 0.055 in/min

SAMPLE DATA:

GRAY CLAY (CL)
DEPTH = 30 ft.
INITIAL DIAMETER = 1.43 in.
INITIAL HEIGHT = 2.91 in.
NATURAL MOISTURE CONTENT = 32%
DRY DENSITY = 92 pcf

Figure E-2
Unconfined Compression Test
B-5 SS-5
Eagle River Dam Project



RATE OF STRAIN = 0.055 in/min

SAMPLE DATA:

GRAY SILT (ML)
DEPTH = 30 ft.
INITIAL DIAMETER = 1.39 in.
INITIAL HEIGHT = 2.81 in.
NATURAL MOISTURE CONTENT = 32%
DRY DENSITY = 93 pcf

Figure E-3
Unconfined Compression Test
B-6 SS-4
Eagle River Dam Project